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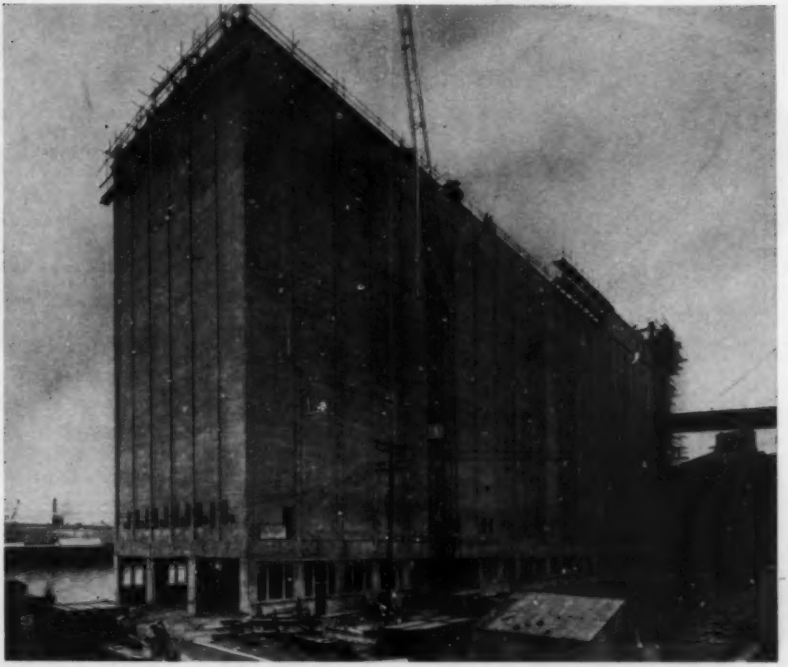

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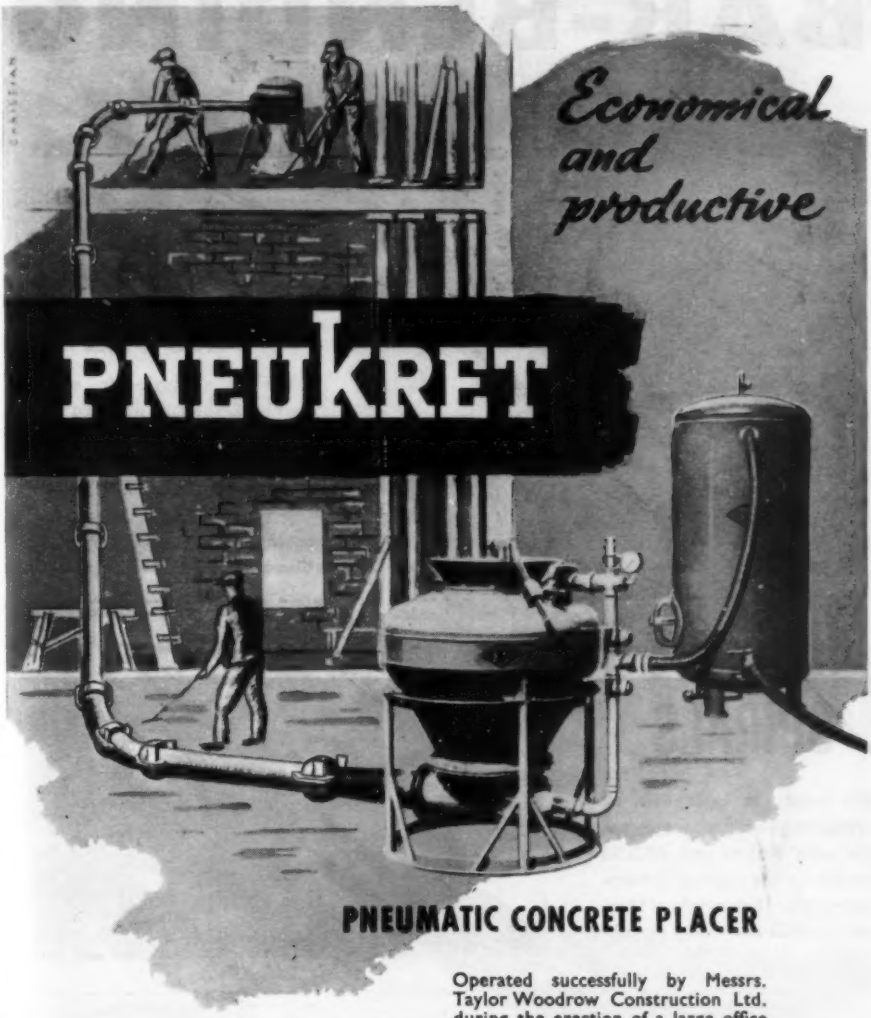

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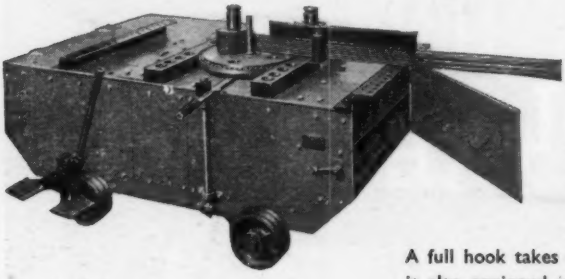

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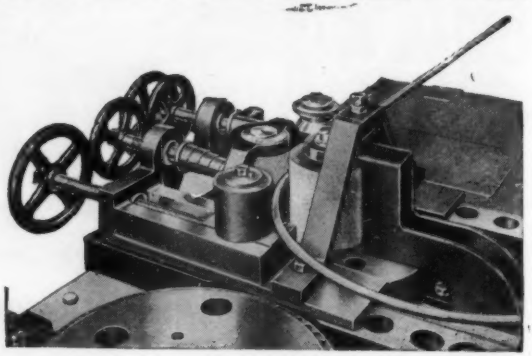

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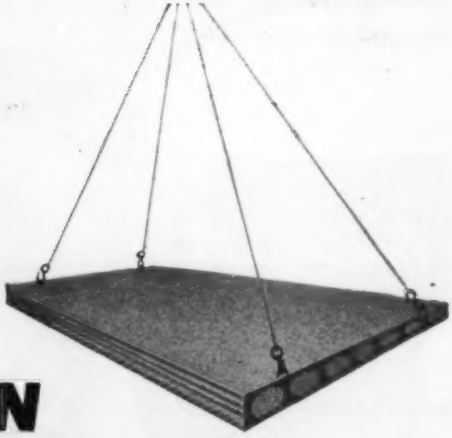

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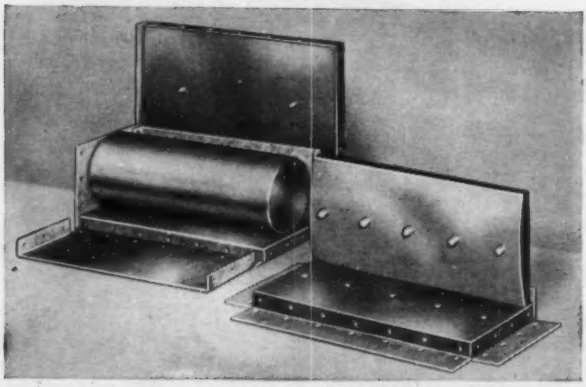

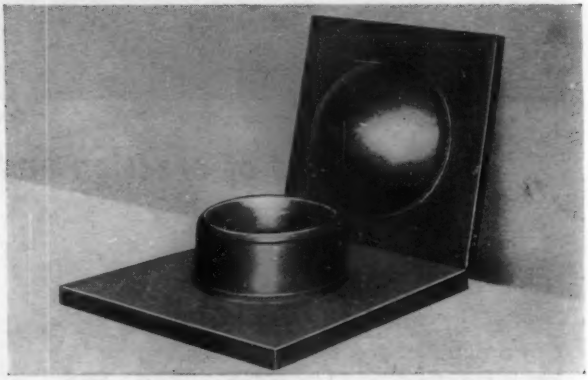

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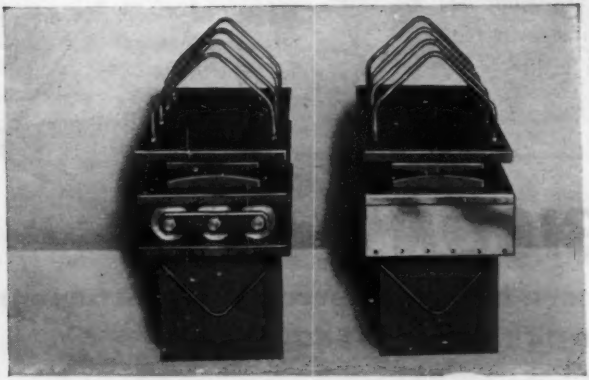

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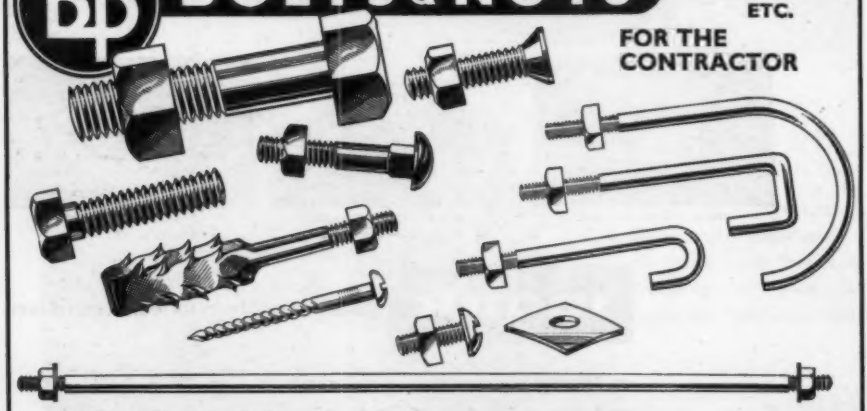

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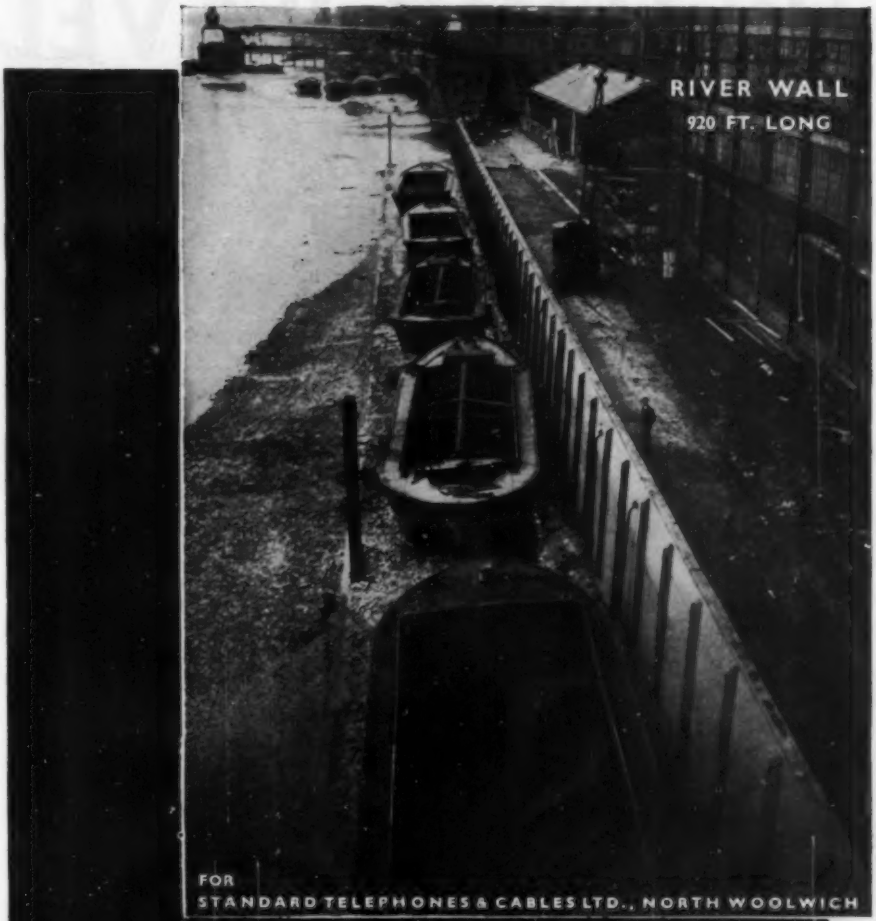

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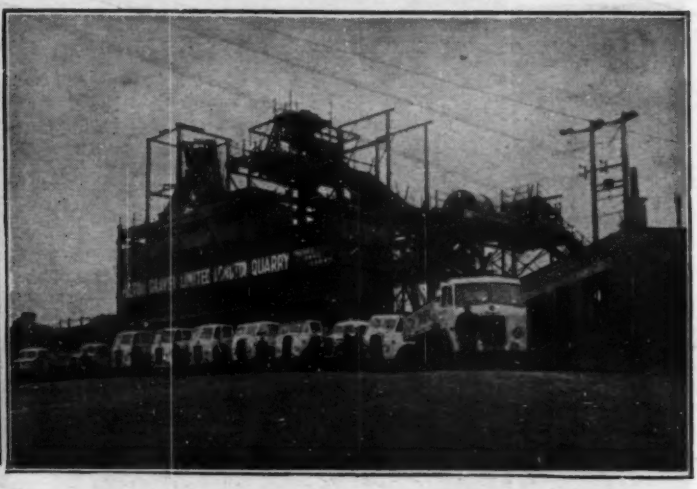

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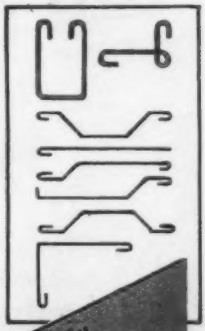

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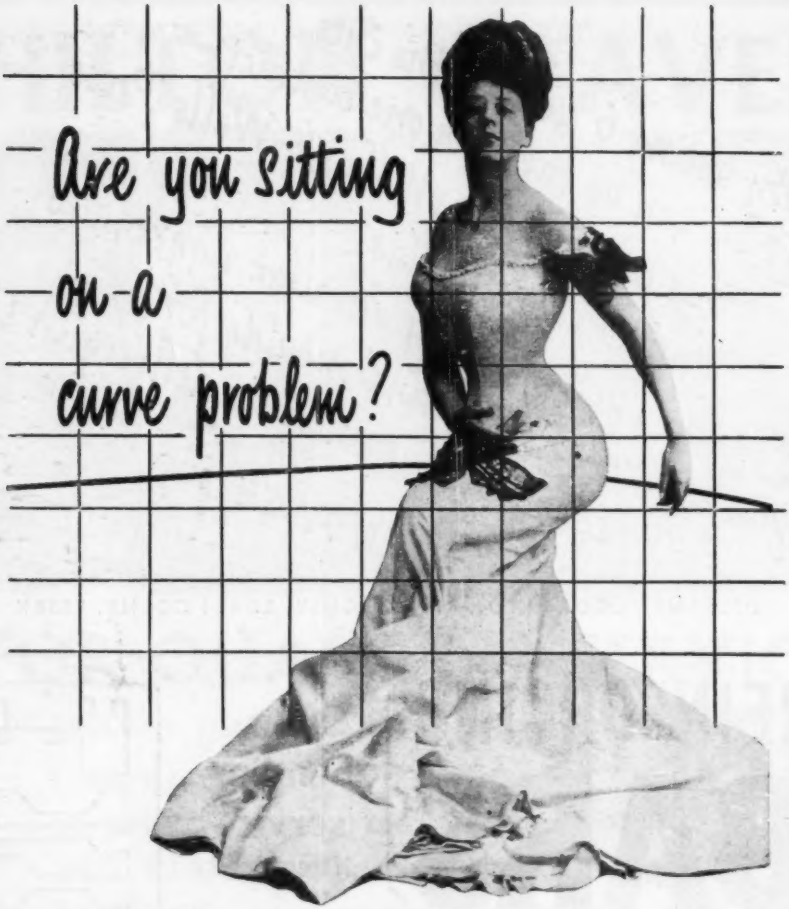

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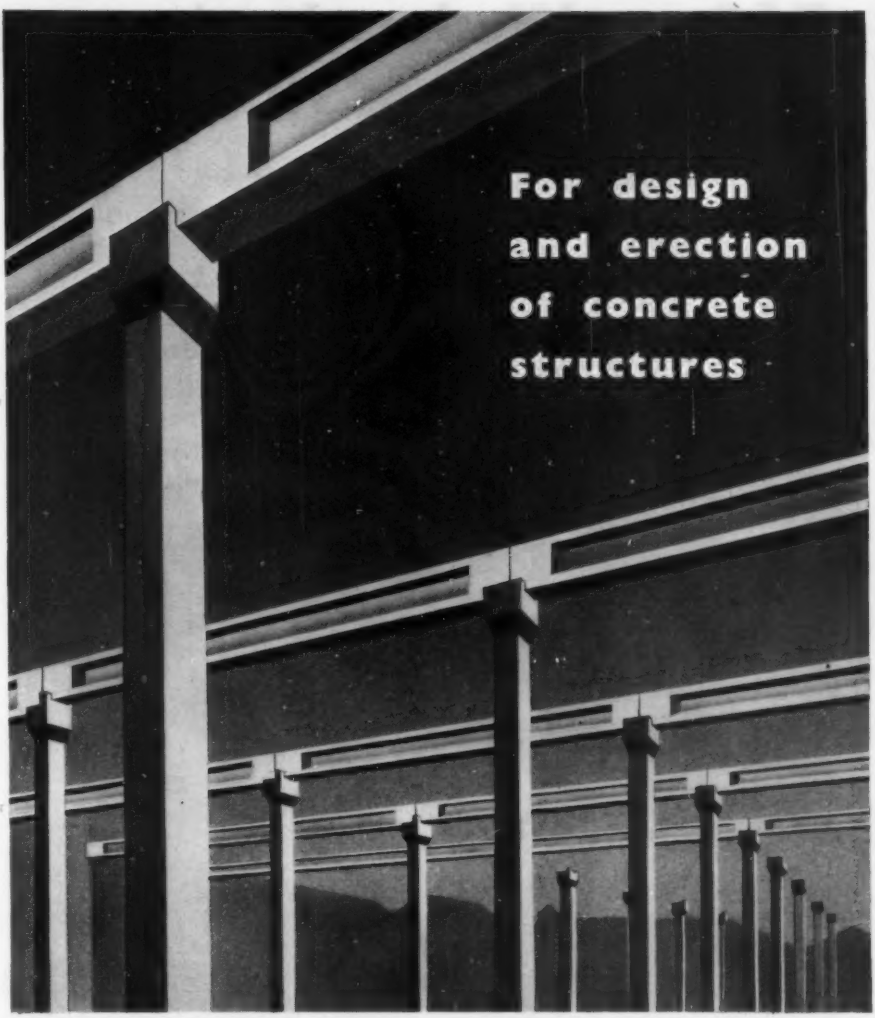

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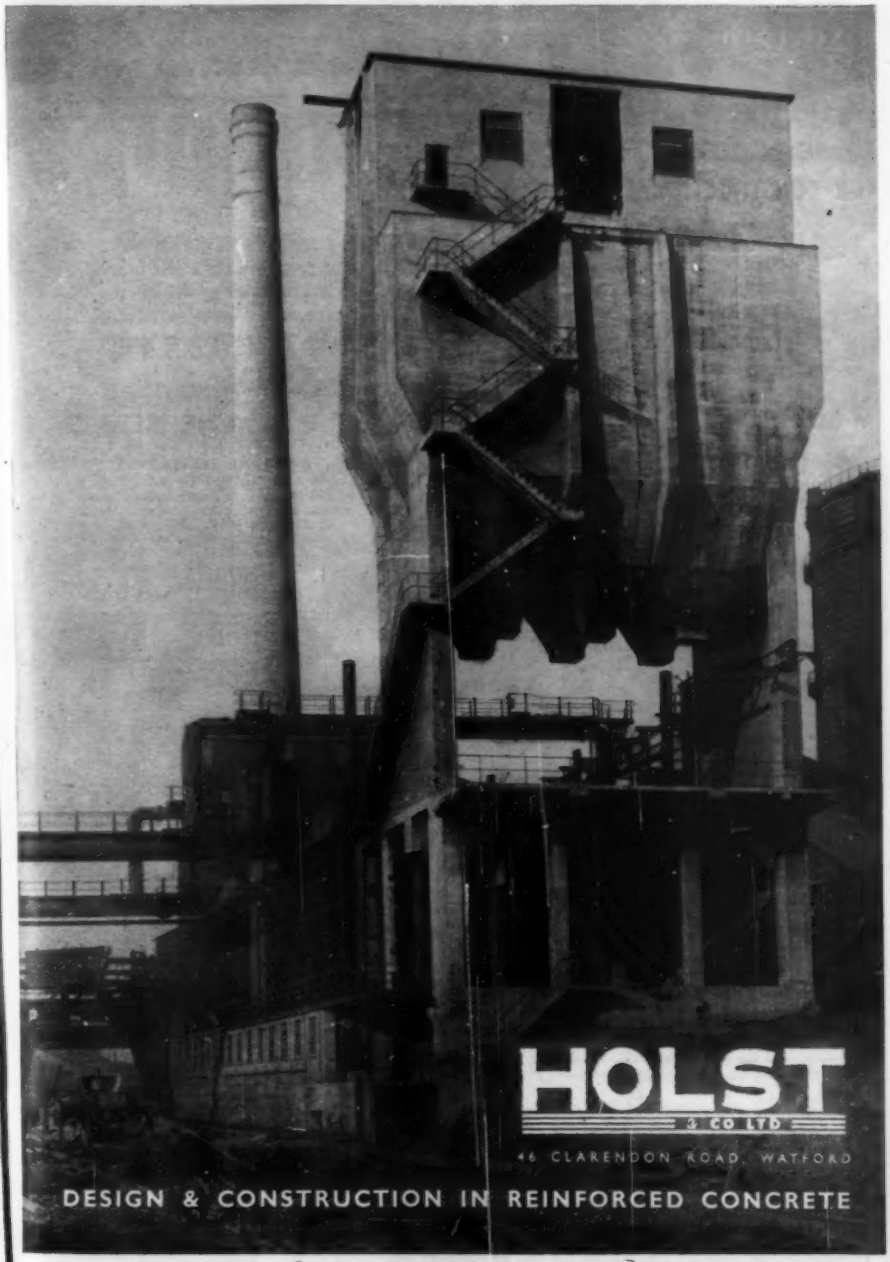

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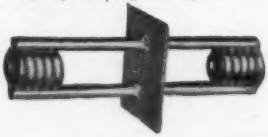

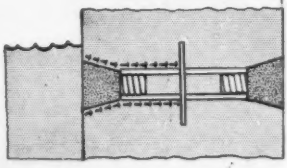

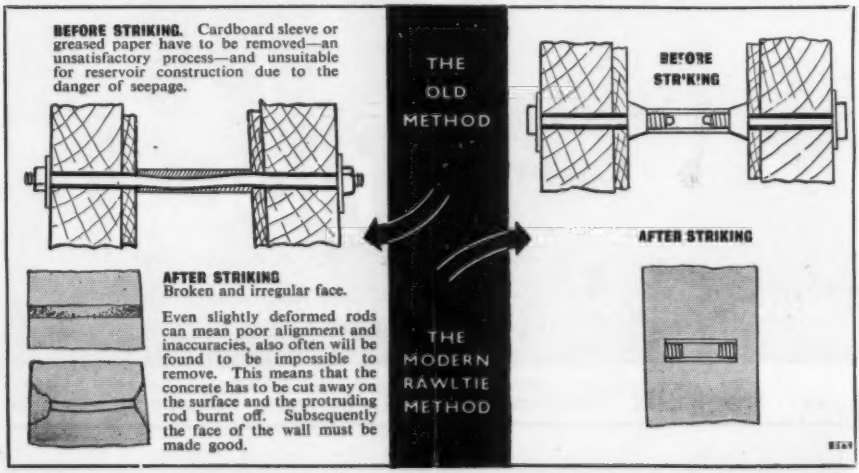
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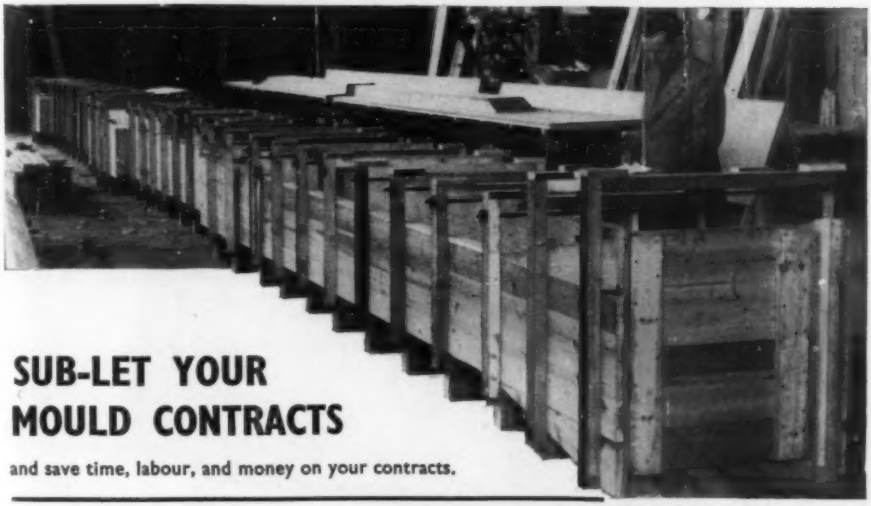

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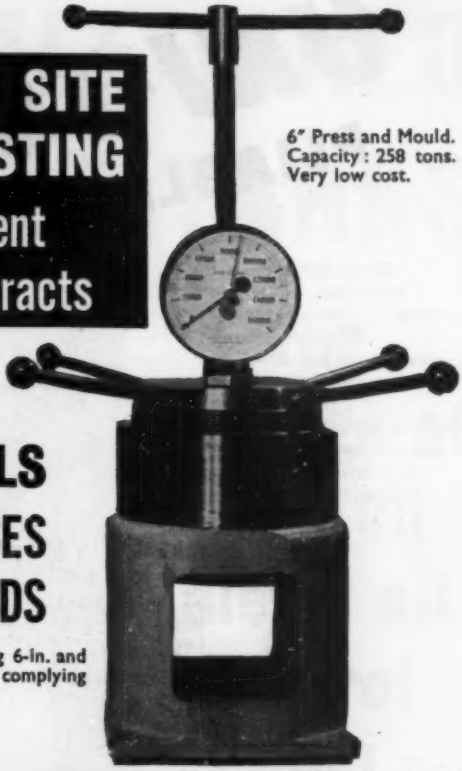
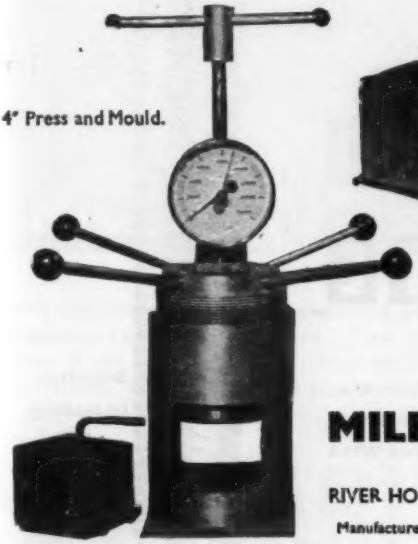


6" Press and Mould.
Capacity: 258 tons.
Very low cost.

MILBANK - WELLS HYDRAULIC PRESSES AND CUBE MOULDS

Presses and moulds in the two sizes for testing 6-in. and 4-in. cubes up to 16,000 lb. per square inch, complying with B.S. 1881.



4" Press and Mould.

Essential for Prestressed Concrete Work

in use all over the world for concrete testing

Full details and prices sent on request.

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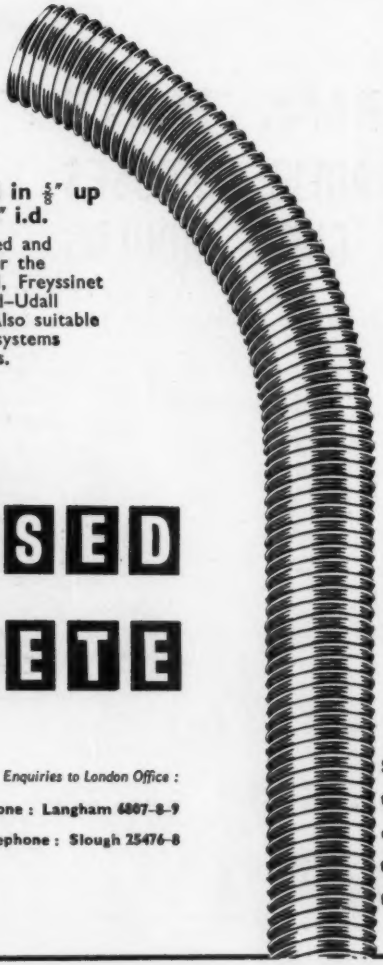
PRESTRESSED CONCRETE CONSTRUCTION

RIVER HOUSE, EARLS COLNE, ESSEX. Tel. : Earls Colne 410

Manufactured for us by Horner & Wells, Ltd., Engineers, Chelmsford, Essex.

UNI-TUBE
PLIABLE STEEL TUBING

Ducts for stressed concrete . . . labour saving . . . easy to install . . . outer ribbing gives a perfect bond to the surrounding concrete . . . the inside of the tube is smooth to facilitate the passage of bars or cables and allows free flow of grout.



Supplied in $\frac{5}{8}$" up to 6" i.d.

As approved and supplied for the Lee–McCall, Freyssinet and Gifford–Udall systems. Also suitable for other systems and designs.

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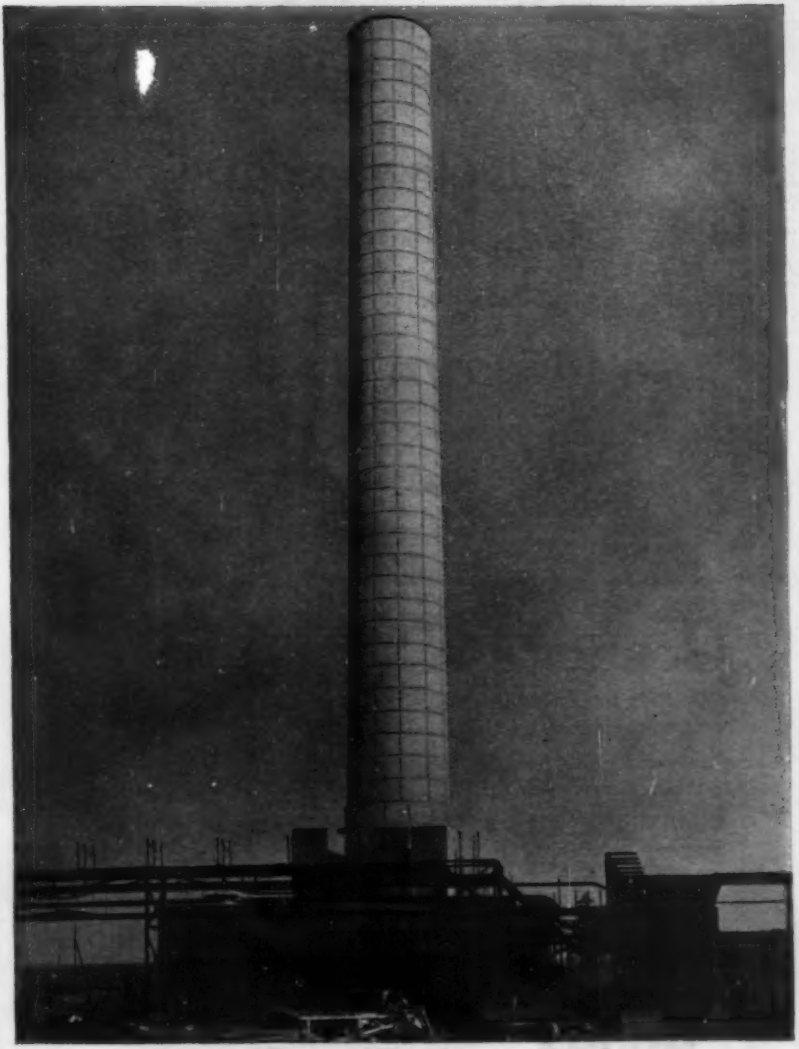

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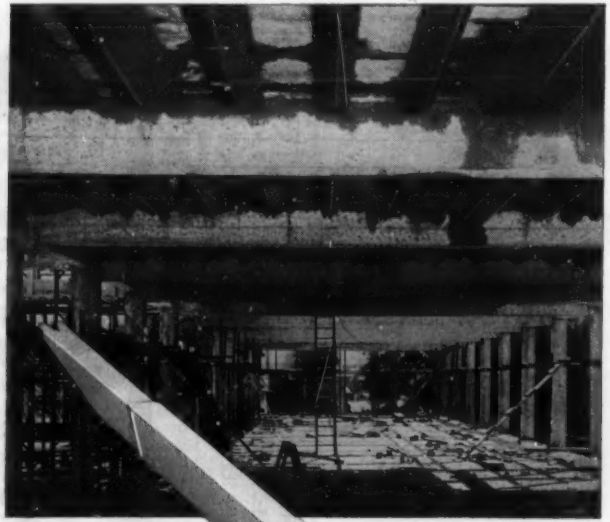

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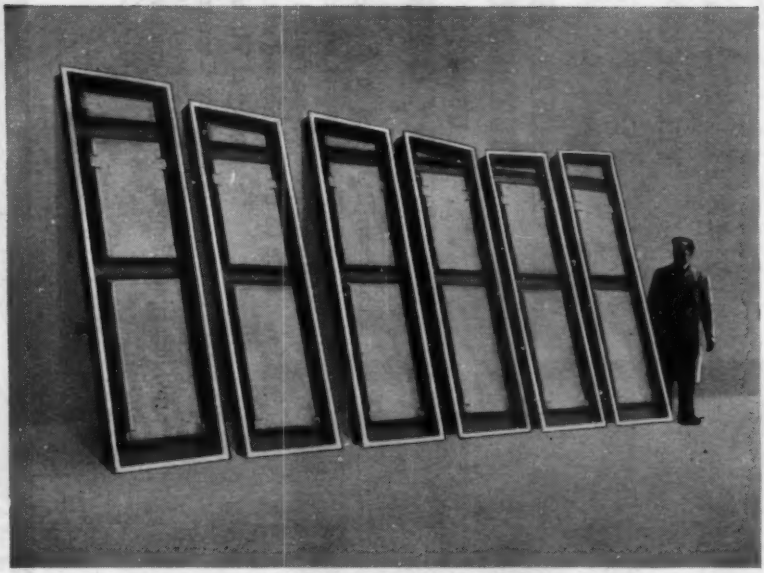

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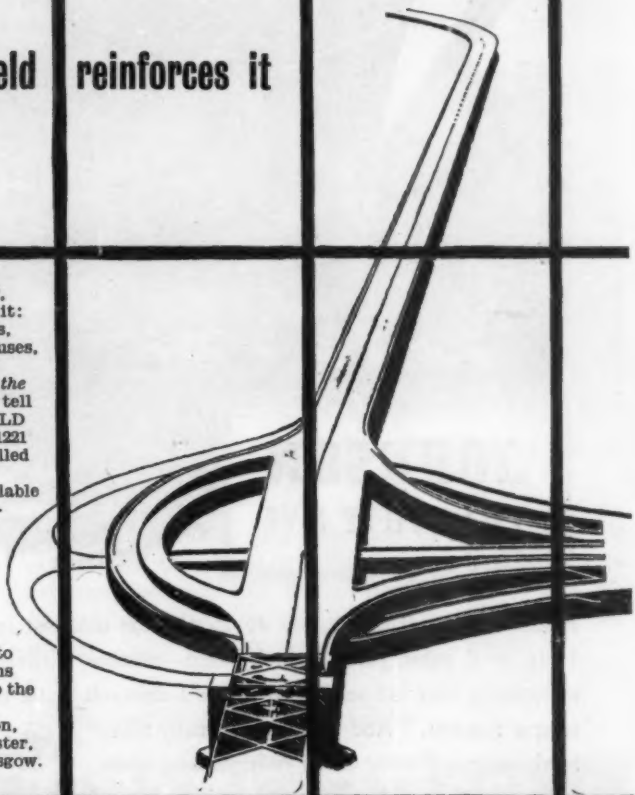
Maxweld reinforces it



Concrete or brickwork, MAXWELD reinforces it: schools, power stations, bridges, roads, warehouses, factories. *Do you need reinforcements? Call up the* MAXWELD *man!* He'll tell you all about MAXWELD fabric—made to BSS. 1221 Part A, closely controlled for quality from raw material onwards, available for quick delivery anywhere. He'll tell you what type you need, how much you need, and how much it's going to cost you. He's backed by *Richard Hill's Design Service*, always ready to work out detailed plans and estimates. Call up the MAXWELD man—at Middlesbrough, London, Birmingham, Manchester, Leeds, Bristol and Glasgow.

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Johnson designed this sturdy dumper as the toughest in its class. It has a 1, 1½ or 2 cubic yards capacity skip—reliable Girling brakes—a chassis embodying four full section rolled-steel channels under the skip—and other unique features. And to ensure it really takes hard usage, a *Rubery Owen fully floating drive axle—manufacturer's rating* 55 *cwt.*—is incorporated. Write for fully illustrated leaflet CC/12/6727 to : —

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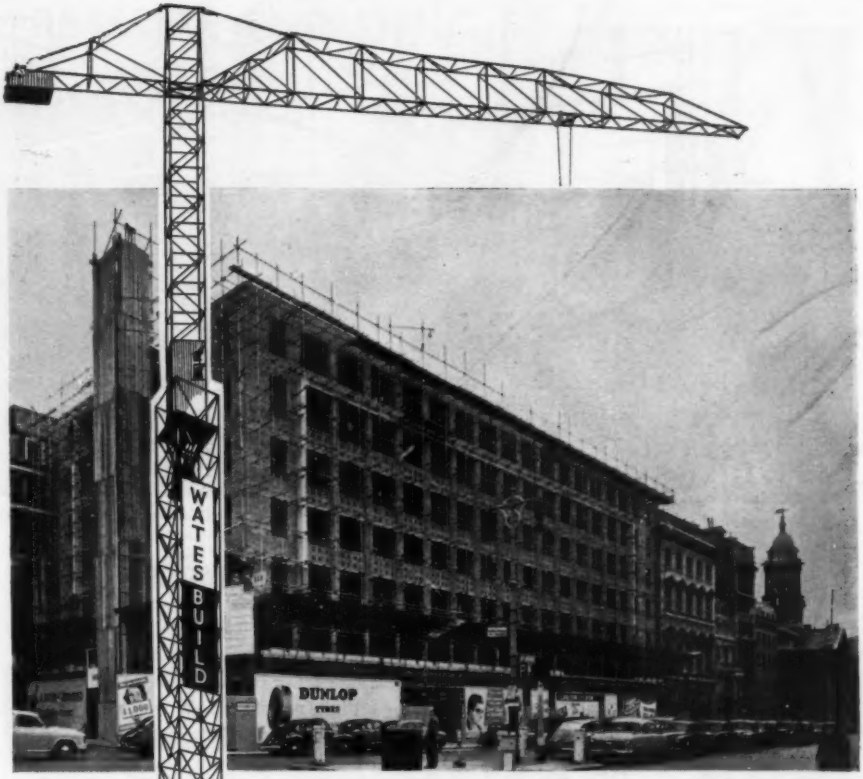

Vogue House, Hanover Square *Architects: Yates, Cook and Darbyshire*

Wates build at high speed and low cost. They have the plant, the organisation and the know-how. Add to this early collaboration with the owner, his architect and his engineer, and the building goes up on time and within the budget. Quick completion means earlier revenue.

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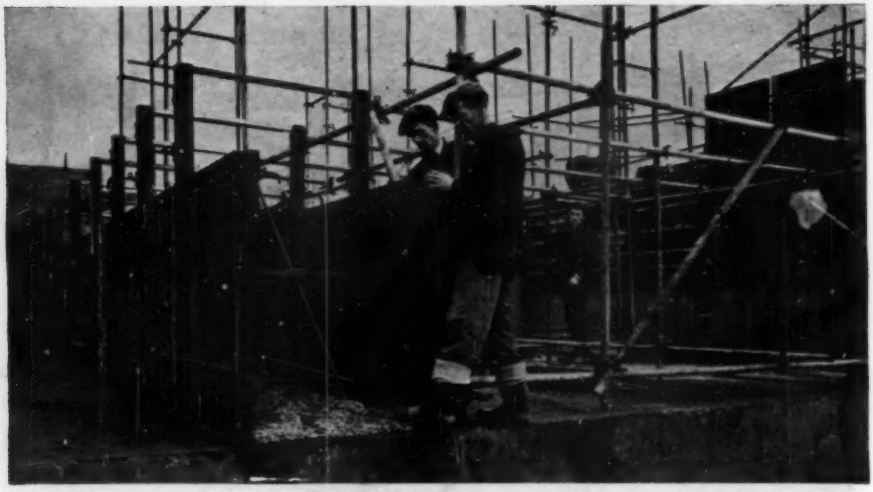


Quoile River Tidal Barrier for Ministry of Agriculture, Government of Northern Ireland

Concrete Facts About Mark II Formwork

Speed and efficiency are essential when you build in reinforced concrete. The Kwikform Mark II Formwork system makes it easy for you to use the most advanced techniques in concrete placing. Mark II needs no external support and will take pours of up to 6 ft. of rapidly deposited concrete—greater pours with the use of the 10'0" long Heavy Duty Aligner. Designed to give the fastest possible speed in fixing and striking, and with only four basic components, the equipment can be handled rapidly and efficiently by unskilled labour. Mark II gives a superior finish. It is adaptable to any dimension—for slab floors, columns, beams and walls. All Kwikform equipment is available on sale or hire terms.

Users of the Mark II system gain the following outstanding advantages:

MATCHLESS *in speed of erection and striking.*

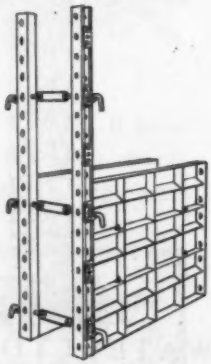
SIMPLE—*only four basic components.*

ADAPTABLE—*designed to meet the widest possible range of uses and to permit the simple use of timber without impairing the speed of erection.*

REVOLUTIONARY *aligning device which completely eliminates the use of cones or distance pieces.*

ECONOMICAL—*fewer loose parts than any other system, with the consequent saving in site losses.*

ELIMINATES *the use of tube and fittings. Walings built-in, giving a further reduction in potential site losses.*



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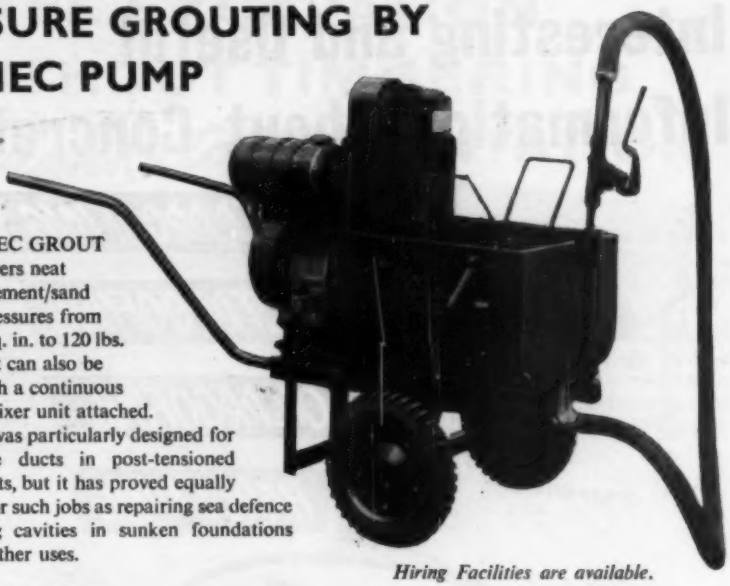
PRESSURE GROUTING BY GALMEC PUMP



The GALMEC GROUT PUMP delivers neat cement or cement/sand grouts at pressures from 10 lbs. per sq. in. to 120 lbs. per sq. in. It can also be supplied with a continuous operation mixer unit attached. This pump was particularly designed for filling cable ducts in post-tensioned concrete units, but it has proved equally successful for such jobs as repairing sea defence walls, filling cavities in sunken foundations and many other uses.

Hiring Facilities are available.

Ask for full particulars from :—

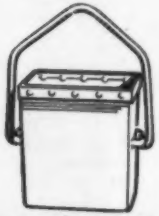
GENERAL MECHANICS LTD., WEST BAY WORKS, BRIDPORT, DORSET

ABELSON *for* SKIPS

Circular tipping skips 2½ cu. ft. to 3 cu. yd.

Concrete pouring skip side discharge 10 cu. ft. to 27 cu. ft.

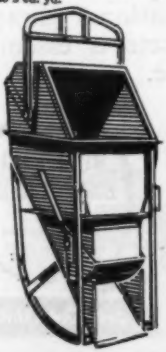
Circular controlled discharge bottom dump skips ¼ cu. yd. to 1 cu. yd.



Square tunnelling or trenching skips 4-fifths cu. ft. capacity.

Concrete pouring skips bottom discharge 10 cu. ft. to 27 cu. ft.

Rollover skips ¼ cu. yd. capacity.



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Colemanoid is a liquid chemical compound which when added to the gauging water will increase the compressive and tensile strength of the concrete by as much as 35%. It will also proof it against oil, water, damp, dust and frost. In certain situations—garages, boiler surrounds, machine shops—oilproof concrete is essential. Colemanoid is your answer—so quick—so effective.

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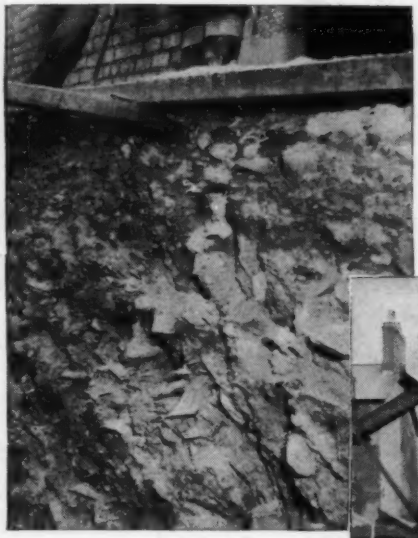

Aberystwyth Borough Council wished to excavate a sewer trench through a very narrow street and within a few feet of houses on each side.

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EXPAMET can take it!

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In those days when the finer techniques of reinforced concrete were being learned, the pioneer spirit of Expanded Metal engineers accepted the challenge to design the reinforcement for the bridge. It has withstood the stress of 50 years and still carries the enormously increased load of modern traffic. During the war it was used by heavy armoured divisions.

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Prompt Delivery. Highly Competitive Prices. Genuine Service. Re-Sale Terms Available.

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Precast Sections Prestressed together by Freyssinet System and Cable Ducts Colgrouted

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Contractors: Christiani & Neilsen Limited.

Mobile Grouting Units with skilled operators are available in the United Kingdom at a fixed daily charge. Upwards of 60 ducts usually grouted per day on site.

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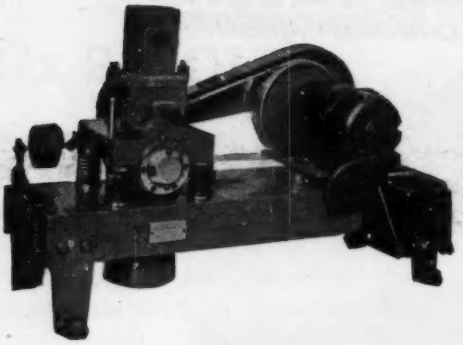

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FORMCRETE at Wimbledon

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The illustration below gives some idea of the area to be covered by the structure, and subsequent advertisements during the next few months will follow the progress of this project until the Spring of 1958, when the building will have been completed.

The Consulting Engineers for this work are Messrs. C. J. Pell and Partners.

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for expansion joints

All Reinforced Concrete Engineers recognise the advantages of using copper strips for sealing joints in concrete work. Copper is ductile, will not crack under repeated bending, is non-corrosive and is unaffected by wet concrete. We specialise in the supply of perforated copper strips of all required lengths and widths for expansion joints, and shall be pleased to submit prices against detailed specification.

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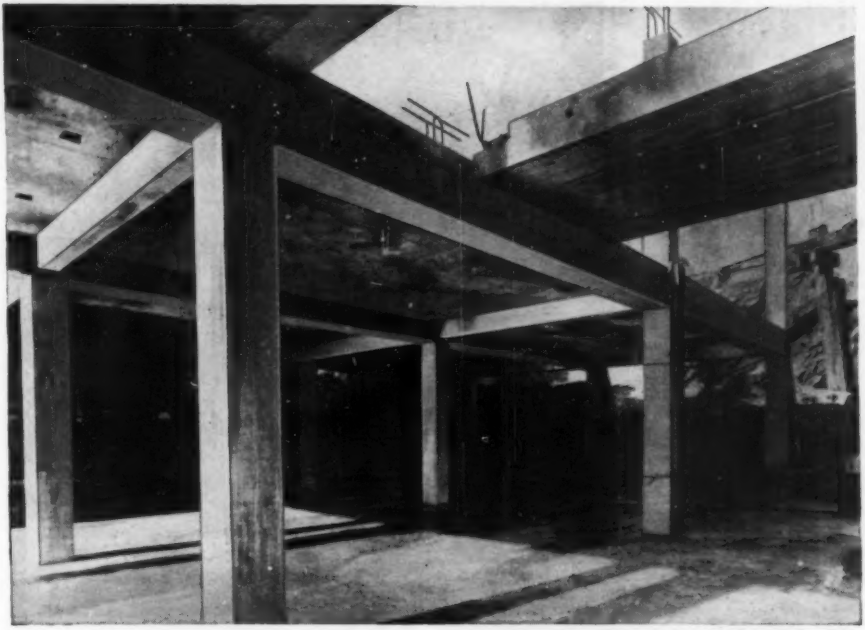

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offers

concrete

advantages

large boards—8′ x 4′ for quick erection

can be used again and again; easily handled

smooth surfaces for best finish

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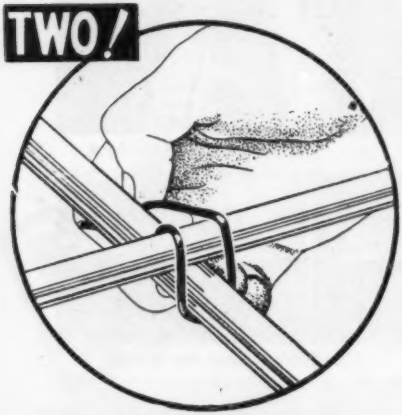

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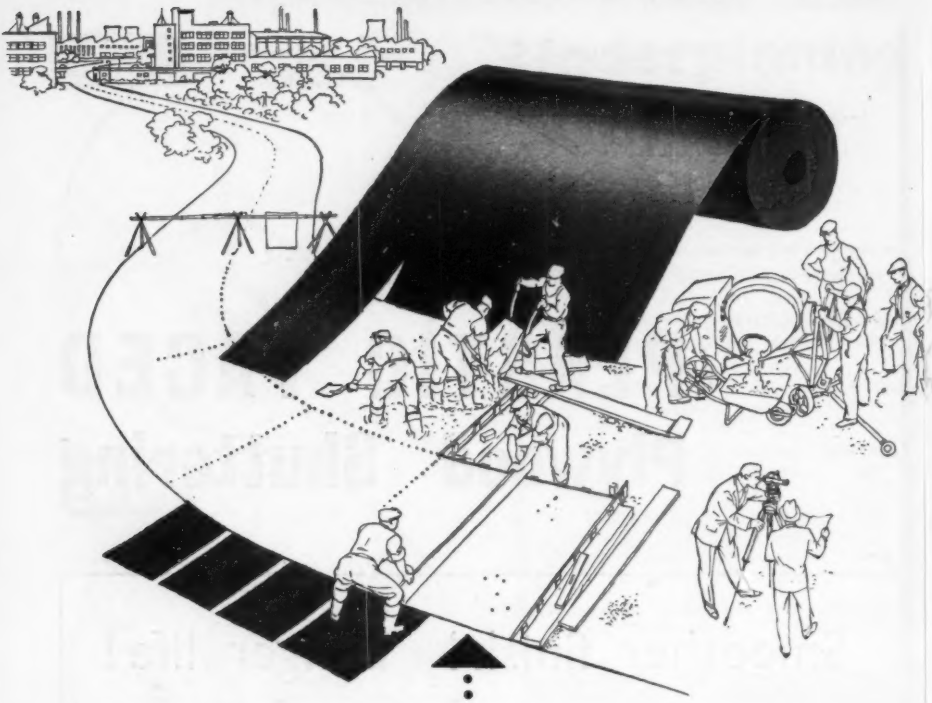

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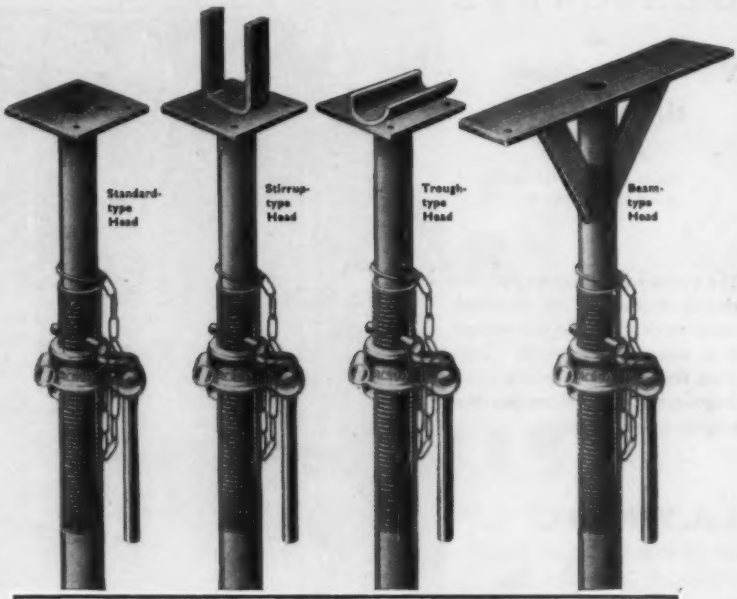

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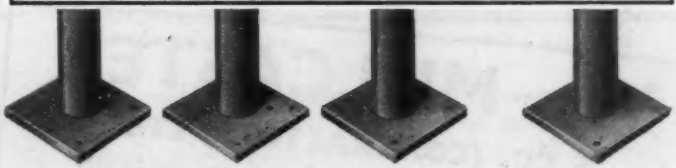

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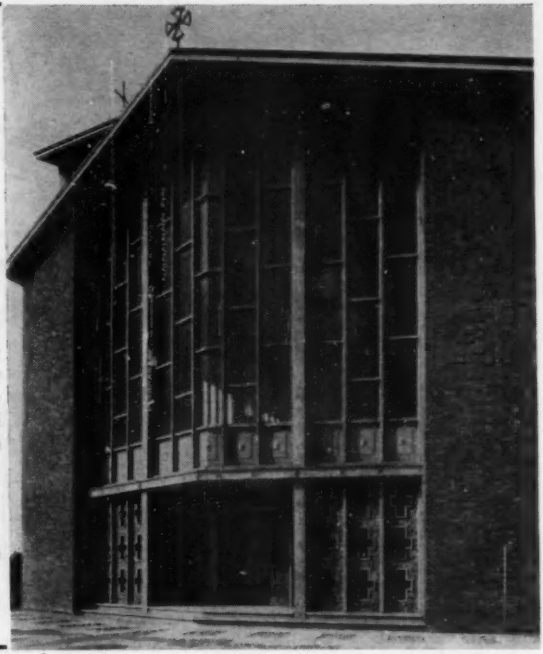

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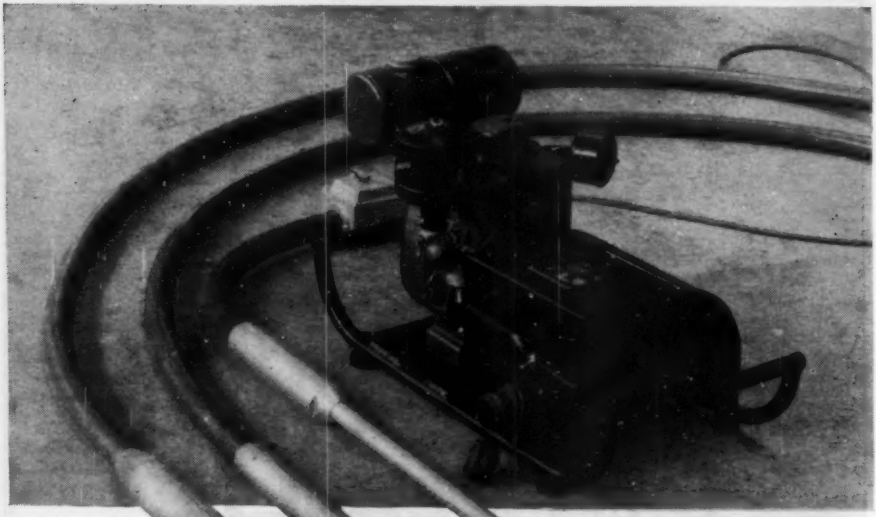

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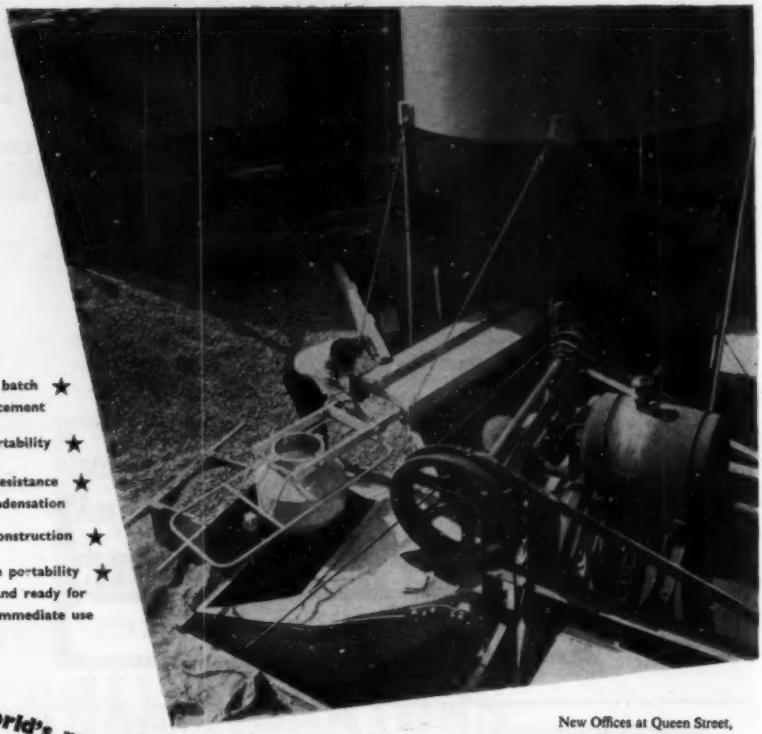

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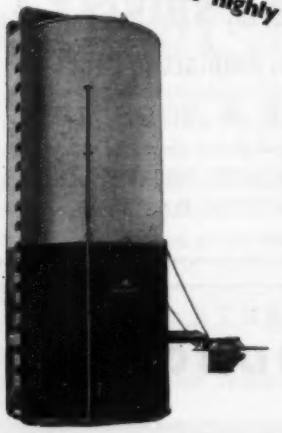

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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Volume LII, No. 10. LONDON, OCTOBER, 1957.

EDITORIAL NOTES

The Importance of First Principles.

IT is a common failing in man that in progressing beyond the elements of any subject he studies he tends to pay less attention to the principles that were first taught to him. Lecturers often find that students who are capable of expressing complex mathematical problems in general terms frequently have difficulty in obtaining correct numerical answers because of elementary mistakes in arithmetic. The failing is not confined to mathematics and engineering, for a former world champion tennis player said that for every ten points scored in championship matches only three were won by unreachable balls while seven were lost by the striker's putting the ball into the net or out of court. Moreover, of the seven points lost four were lost because the striker committed the elementary blunder of taking his eye off the ball. Thus the beaten player fails to achieve his ambition because he forgets the first lesson he received.

The student of reinforced concrete is taught in his first lesson that although the concrete below the steel in the bottom of a beam is assumed not to add to the strength of the beam it nevertheless fulfils a most important function in protecting the steel from corrosion. He is also told that the two materials should act together as one without there being any relative movement between the steel and the concrete. Unfortunately, when the student becomes a designer, the earlier part at least of the first lesson is often forgotten. An examination of faulty reinforced concrete structures, some of them erected less than twenty years ago, shows that the most obvious faults are generally the spalling and splitting of the concrete as a result of corrosion of the reinforcement due to insufficient cover. Sometimes these faults start at the positions of stirrups or hoops because the thickness of cover has been specified for the main bars without allowance being made for the reduction, equal to the diameter of the secondary bars, at these points. Even when sufficient cover is specified a positive means for ensuring the cover by the use of metal chairs or concrete spacers is not always provided and the method of supporting the bars during concreting is left to the ingenuity of the steel fixer. In competitive design there may be a temptation to reduce the size of members, for even a small amount saved on each story of a multiple-story building may enable considerable saving to be made on walls and finishes. If this is done by reducing the cover to the bars it may well prove to

be a false economy, as in the case of a building in the south of England which has recently cost its owners over £20,000 in repairs because of the failure to comply with the elementary principle of protecting the reinforcement from corrosion.

Many of the cracks in concrete structures occur because sufficient attention has not been given to bond stresses and to anchoring of the reinforcement. In recent years more care has been taken to avoid the use of discontinuous systems of reinforcement and so to reduce the concentrations of stress which lead to the appearance of cracks. Nevertheless in some designs, particularly at the joints between beams and columns where the reinforcement is congested and the column bars are both cranked and lapped, there is a likelihood of high bond stresses occurring with the possibility of relative movement between the concrete and the steel. The difficulty of obtaining adequate bond strength is often accentuated by insufficient distance being allowed horizontally between separate bars and vertically between layers of bars. Not only may this result in a lack of adhesion between the concrete and the steel but, by preventing the concrete being properly compacted between the bars, it may lead to inadequate protection being given to the bars. The present codes of practice recommend that there shall be a distance of at least ½ in. vertically between bars lying in a horizontal direction. Compliance with the least requirement is, however, frequently considered to be all that is necessary, and it would seem desirable to increase this distance to ¾ in. in beams with closely-spaced reinforcement, if not in all beams.

In the first chapter of many text-books on reinforced concrete mention is made of the shrinkage of concrete and its thermal expansion and contraction. Too frequently that is the only place in which these are mentioned, and designers have been known to overlook the forces that result when these movements are restrained. If the foundations of a building are fixed and the floors expand or contract stresses are produced in the columns due to their resistance to bending, and on occasions these stresses are greater than those due to the loads. It is, unfortunately, usually on the most prominent features of a building that the worst damage occurs, and balconies are particularly liable to injury by this cause. To guard against thermal movement seems an elementary precaution, but it is another of the first principles that are sometimes forgotten.

"Keeping one's eye on the ball" while engaged in the analysis of forces and moments is largely a matter of keeping the structure in mind and not allowing intricate, although no doubt interesting, mathematical devices to confuse the problem rather than to clarify it. A well-known continental engineer said that "simplicity is more worth striving after than the greatest precision", but it is difficult to believe that this precept is followed in all the methods advocated for the solution of problems in structural engineering. Some of these methods are attempts to save time in the drawing office by reducing design to the solution of equations which require only the insertion of dimensions and loads. Often the result is that by avoiding consideration of the problem the method is applied to conditions for which it was not intended. In attempting to reduce the solution of problems to the mechanical manipulation of formulæ, which the user infrequently derives or checks for himself, the structural intuition of the designer can be dimmed and the first principles of statics become so hidden that an improbable answer may be accepted unthinkingly.

Bending Moments on Continuous Beams in Accordance with the Revised Code.

By CHAS. E. REYNOLDS, B.Sc.(Eng.), A.M.I.C.E.

THE revised British Standard Code of Practice No. 114 : 1957 recommends, in effect, three methods of calculating the bending moments on slabs spanning in one direction and on beams continuous over a number of supports : (1) If the spans are nearly equal and the loads are uniformly distributed, from simple approximate expressions. (2) From theoretical analyses assuming that the slabs or beams are free to rotate about the supports ; that is, the common assumption of knife-edge supports. (3) Or beams may be considered to be part of a monolithic frame. The application of these methods is considered here since in the Code there are important modifications (generally simplifications) compared with the recommendations in the preceding code (B.S. Code No 114 : 1948 and subcodes of 1951).

Equal Spans : Approximate Method.

For slabs spanning in one direction and beams continuous over three or more equal spans and carrying uniformly-distributed loads, the coefficients based on the expressions in the revised Code for the calculation of the bending moments due to imposed and dead loads are given in *Table* I, which also gives the combined

TABLE I.—CONTINUOUS BEAMS : EQUAL SPANS. APPROXIMATE BENDING MOMENT COEFFICIENTS. UNIFORMLY-DISTRIBUTED LOADS.

	Dead load only	Live load only	Combined dead and live load : $R = \frac{\text{live load}}{\text{dead load}}$			
			$R = \frac{1}{2}$	1	2	3
End span : about midspan	$+\frac{1}{12}$	$+\frac{1}{10}$	$+\frac{1}{11{\cdot}2}$	$+\frac{1}{10{\cdot}9}$	$+\frac{1}{10{\cdot}6}$	$+\frac{1}{10{\cdot}4}$
Penultimate support	$-\frac{1}{10}$	$-\frac{1}{9}$	$-\frac{1}{9{\cdot}6}$	$-\frac{1}{9{\cdot}5}$	$-\frac{1}{9{\cdot}3}$	$-\frac{1}{9{\cdot}2}$
Interior spans : midspan	$+\frac{1}{24}$	$+\frac{1}{12}$	$+\frac{1}{17{\cdot}8}$	$+\frac{1}{16}$	$+\frac{1}{14{\cdot}4}$	$+\frac{1}{13{\cdot}7}$
Interior supports	$-\frac{1}{12}$	$-\frac{1}{9}$	$-\frac{1}{10{\cdot}8}$	$-\frac{1}{10{\cdot}3}$	$-\frac{1}{9{\cdot}8}$	$-\frac{1}{9{\cdot}6}$

Bending Moment = (coefficient) × (total load) × (span).

coefficients for the total bending moment at the critical sections for various ratios of live to dead loads. To obtain the bending moments, the coefficients are multiplied by the product of the intensity of uniformly-distributed load and the square of the span. Consecutive spans are considered to be equal if the difference

TABLE II.

CONTINUOUS BEAMS
BENDING MOMENTS

TWO & THREE EQUAL SPANS
UNIFORM MOMENT OF INERTIA

TWO SPANS		THREE SPANS	
DEAD LOAD (ALL SPANS LOADED)	LIVE LOAD	DEAD LOAD (ALL SPANS LOADED)	LIVE LOAD
0·125 (0·106)	0·125 (0·106)	0·100 (0·085)	0·117 (0·100)
	0·063 [0·072]		0·050 [0·058]
0·070 (0·080)	0·096 [0·091]	0·080 (0·088)	0·101 [0·098]
		0·025 (0·040)	0·017 (0·015)
			0·075 [0·067]
0·187 (0·159)	0·187 (0·159)	0·150 (0·128)	0·175 (0·149)
W	W	W	W
W	0·094 [0·108]	W	0·075 (0·086)
	W	W	W
0·156 (0·170)	0·203 [0·196]	0·175 (0·186)	0·025 (0·021)
		0·100 (0·123)	0·213 [0·207]
			0·175 [0·164]
0·167 (0·142)	0·167 (0·142)	0·134 (0·114)	0·156 (0·133)
W/2 W/2 W/2 W/2	W/2 W/2 W/2 W/2	W/2 W/2 W/2 W/2	W/2 W/2 W/2 W/2
	0·083 [0·095]		[0·077] 0·067
0·111 (0·119)	0·139 [0·135]	0·122 (0·129)	0·142 [0·139]
0·056 (0·073)	0·111 [0·103]	0·075 (0·091)	0·022 (0·019)
		0·066 [0·086]	0·122 [0·116]
			0·100 [0·090]

EQUAL LOAD W (TOTAL LOAD) ON EACH LOADED SPAN.
BENDING MOMENT = (COEFFICIENT) x (W) x (SPAN)
DIAGRAMS ARE SYMMETRICAL BUT NOT DRAWN TO SCALE.

COEFFICIENTS THUS: 0·125 = THEORETICAL BENDING MOMENTS
(0·106) = BENDING MOMENTS WITH B. Ms AT SUPPORTS REDUCED BY 15 PER CENT. (GIVE ADJUSTED B.Ms. FOR DEAD LOAD AND REDUCED SUPPORT B.Ms. FOR LIVE LOAD)
[0·191] = BENDING MOMENTS WITH B. Ms AT SUPPORTS INCREASED BY 15 PER CENT. [GIVE REDUCED MID-SPAN B.Ms. FOR LIVE LOAD]

in length does not exceed 15 per cent. of the longer span. The coefficients are based on the assumption that the slab or beam is freely supported on the outer supports. Since this condition is rarely obtained in the case of solid slabs, and since the torsional resistance of the supporting beams is generally effective in restraining a slab from acting freely as if it were carried on "knife-edge" supports, it is questionable whether it is necessary to be so precise in the estimation of the bending moments on solid slabs as is indicated by the Code. It is thought that coefficients of one-tenth for an end span and penultimate support and one-twelfth for an interior span and support, which have been adopted for many decades, are sufficiently realistic for such slabs.

Theoretical Analysis : "Knife-edge" Support.

If the spans are unequal or the loads are not uniformly-distributed, or if both these cases apply, the bending moments can be calculated in accordance with one of the exact theoretical methods, such as the theorem of three moments, slope-deflection, strain energy, and the theorem of least work. This procedure is generally complex, even when permissible simplifying assumptions are made ; less exact methods, such as the displacement method, ultimate-load (plastic) method, and moment-distribution and variants thereof, are simpler and sufficiently accurate. An application of moment distribution allowing for all variations of span, load, and moment of inertia is given in this journal for December, 1948.

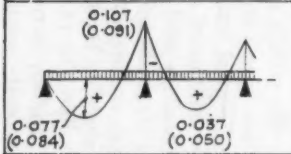
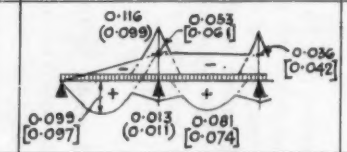
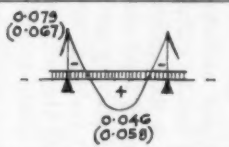
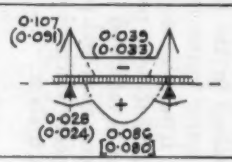
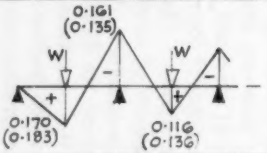
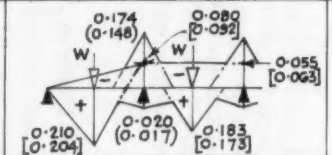
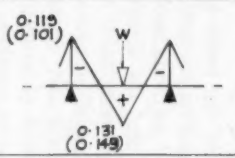
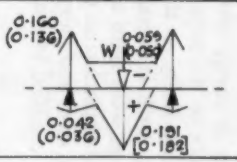
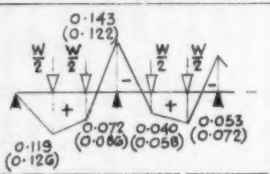
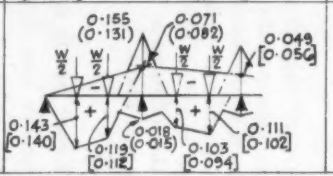
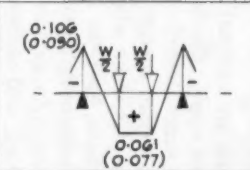
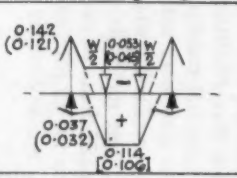
When a strictly theoretical analysis is adopted, the Code recommends that the bending moments over the supports, calculated for any arrangement of load, may be increased or decreased by not more than 15 per cent. if the bending moments on the adjacent spans, and due to the same arrangement of load, are based on these modified bending moments. The application of this rule is considered in the following in connection with beams and slabs continuous over a number of equal spans.

Equal Spans and Uniform Moment of Inertia.

For the common case of beams continuous over two or more spans and of uniform moment of inertia throughout all spans, and freely supported at the end supports, it is worth while tabulating the bending-moment diagrams and the coefficients for the bending moments near the middle of each span and at each support as shown in *Table* II (two and three spans) and *Table* III (four or more spans) for a uniformly-distributed load, a central load, and loads concentrated at the third-points of each span. The condition of all spans loaded (for example, dead load) and conditions of incidental (or live) load producing the greatest bending moments are considered. As the coefficients are calculated by the theorem of three moments (and are therefore not approximate), the decrease or increase of 15 per cent. of the maximum negative bending moments at the supports is permissible. These coefficients are reduced by 15 per cent. to establish the reduced bending moments at the supports, and are increased by 15 per cent. to derive the reduced positive bending moments on the spans. *Tables* II and III also give the coefficients for the positive bending moments at the supports and

TABLE III.

CONTINUOUS BEAMS — FOUR OR MORE EQUAL SPANS
BENDING MOMENTS — UNIFORM MOMENT OF INERTIA

END SPAN AND PENULTIMATE SPAN & SUPPORT (BASED ON FOUR CONTINUOUS SPANS)		OTHER INTERIOR SPANS & SUPPORTS (BASED ON CENTRAL SPAN OF FIVE CONTINUOUS SPANS)	
DEAD LOAD (ALL SPANS LOADED)	LIVE LOAD	DEAD LOAD (ALL SPANS LOADED)	LIVE LOAD
			
			
			

EQUAL LOAD W (= TOTAL LOAD) ON EACH LOADED SPAN.
BENDING MOMENT = (COEFFICIENT) X(W) X(SPAN)
COEFFICIENTS THUS: 0·116 = THEORETICAL BENDING MOMENTS.
(0·099) = BENDING MOMENTS WITH B.MS. AT SUPPORTS REDUCED BY 15 PER CENT. (GIVE ADJUSTED B.MS. FOR DEAD LOAD AND REDUCED SUPPORT B.MS. FOR LIVE LOAD)
[0·097] = BENDING MOMENTS WITH B.MS. AT SUPPORTS INCREASED BY 15 PER CENT. [GIVE REDUCED MID-SPAN B.M.S. FOR LIVE LOAD]

DIAGRAMS ARE NOT DRAWN TO SCALE.
DIAGRAMS FOR "OTHER INTERIOR SPANS & SUPPORTS" ARE SYMMETRICAL.

the negative bending moments on the spans which are produced under some conditions of live load ; it is not generally necessary to take these small bending moments into account as they are insignificant compared with the bending moments due to dead load.

The method of calculating the adjusted coefficients in *Tables* II and III is shown by the example in *Fig.* 1 of a beam continuous over three spans and loaded with dead and live loads concentrated at midspan. The theoretical bending moments are calculated for (i) all spans loaded (dead load), and for each of the four cases of live load that produce maximum bending moments, that is at (ii), the middle of an end span (positive) ; (iii) an inner support (negative) ; (iv) the middle of the central span (positive) ; and (v) an inner support (positive). For each case, the theoretical bending-moment diagram is adjusted as described in the following. For the diagram of maximum negative bending moments, the theoretical negative bending moments at the supports are reduced by 15 per cent. and the positive bending moments are increased accordingly. For the diagram of maximum positive bending moments in the spans, the theoretical negative bending moments at the supports are increased by 15 per cent. and

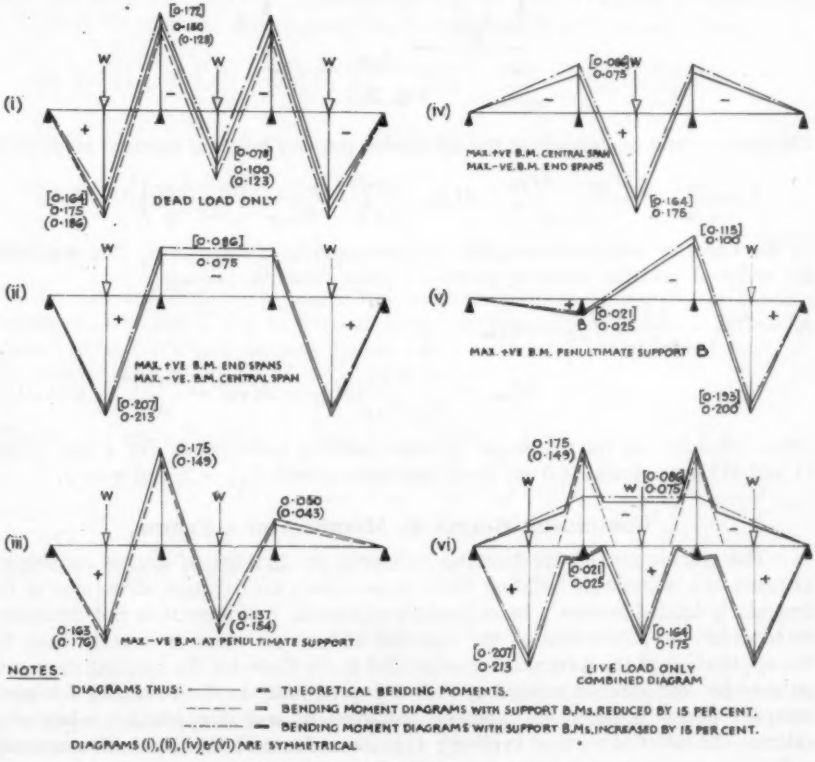


Fig. 1.

the positive bending moments are reduced accordingly. The diagrams for live load are combined in (vi) to give the resultant diagrams for the theoretical bending moments without adjustment and with the greatest degree of adjustment. The maximum bending moments given in *Fig.* 1 are for unit total load W on unit span, and are therefore the bending moment coefficients in *Table* II (three spans) for a load applied at the centre of the span.

The position and magnitude of the maximum positive bending moments are readily calculated for beams carrying concentrated loads. For a beam such as ST of span L_{ST} in *Fig.* 2, carrying a uniformly-distributed load of intensity w, and subjected to theoretical negative bending moments M_{ST} at S and M_{TS} at T,

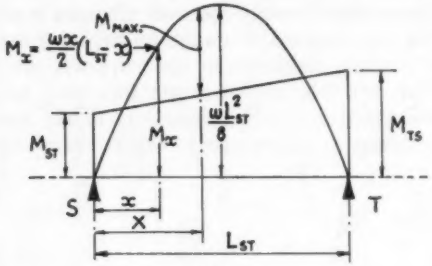


Fig. 2.

the position and magnitude of the maximum positive bending moment is given by

$$X = \frac{L_{ST}}{2} + \frac{M_{ST} - M_{TS}}{wL_{ST}}; \quad M_{max.} = \frac{w}{2}\left(\frac{M_{ST} - M_{TS}}{wL_{ST}} + \frac{L_{ST}}{2}\right)^2 - M_{ST}.$$

If the negative bending moments are increased by 15 per cent. (the condition for reduced positive bending moment) these formulæ become

$$X' = \frac{L_{ST}}{2} + \frac{1{\cdot}15}{wL_{ST}}(M_{ST} - M_{TS});$$

$$M'_{max.} = \frac{w}{2}\left[\frac{1{\cdot}15}{wL_{ST}}(M_{ST} - M_{TS}) + \frac{L_{ST}}{2}\right]^2 - 1{\cdot}15\,M_{ST}.$$

The coefficients for the maximum positive bending moments in *Fig.* 1 and *Tables* II and III are calculated from these expressions with $L_{ST} = 1$ and $w = 1$.

Continuous Beams as Members of a Frame.

The calculation of the bending moments on continuous beams considered as parts of a monolithic building frame is necessary for irregular structures or for irregularly-loaded beams. In ordinary rectangular buildings it is not necessary to consider the interaction of the columns and beams more accurately than by the application of the formulæ recommended in the Code for the bending moments on exterior and interior columns. A less approximate method suitable for more complex beams is based on moment distribution, and is applicable when considering the effect of vertical loading; the effect of lateral deformation (commonly called " sway ") is excluded; the method requires amplification if large out-of-

balance loads are present, and it is not applicable to investigation of the effects of wind which, however, can be dealt with simply. The method is similar to that described in this journal for December, 1948, for continuous beams on knife-edge supports, but in the present treatment the stiffness of the supports is taken into account.

The convention of signs is that bending moments producing deformation of the beam such that it is " concave downwards " are considered to be negative and bending moments producing the " concave upwards " condition are positive, thereby conforming to common practice as regards beams but differing from the convention usually adopted for frames. Thus, the fixed-end bending moments are negative at both ends of the span of a beam.

Consider any interior span ST in a series of continuous spans QR, RS, ST, TU, UV in the general rectangular frame in *Table* IV. The stiffness factors $K = \frac{I}{L}$ for each of the members meeting at S and T are, for beam RS, $K_{RS} = \frac{I_{RS}}{L_{RS}}$, for column SO, $K_{SO} = \frac{I_{SO}}{L_{SO}}$, etc., as given in the table. The sum of the distribution factors for the four members meeting at S is unity (and likewise unity at T), and the separate distribution factors in terms of stiffness factors are

$$\text{At S.—}D_{SR} = \frac{K_{RS}}{\Sigma K_S};\quad D_{SP} = \frac{K_{SP}}{\Sigma K_S};\quad D_{ST} = \frac{K_{ST}}{\Sigma K_S};\quad D_{SO} = \frac{K_{SO}}{\Sigma K_S},$$

in which $$\Sigma K_S = K_{RS} + K_{SP} + K_{ST} + K_{SO}.$$

$$\text{At T.—}D_{TS} = \frac{K_{ST}}{\Sigma K_T};\quad D_{TX} = \frac{K_{TX}}{\Sigma K_T};\quad D_{TU} = \frac{K_{TU}}{\Sigma K_T};\quad D_{TY} = \frac{K_{TY}}{\Sigma K_T},$$

in which $$\Sigma K_T = K_{ST} + K_{TX} + K_{TU} + K_{TY}.$$

By two distribution operations and one intermediate carry-over, the bending moments at S and T due to vertical loads on spans RS, ST, and TU (loads on spans QR and UV and beyond do not affect S and T in this limited moment-distribution operation) are

$$M_{ST} = -F_{ST} + D_{ST}(F_{ST} - F_{SR}) + \frac{D_{TS}}{2}(F_{TU} - F_{TS}) - D_{ST}\left[\frac{D_{TS}}{2}(F_{TU} - F_{TS}) + \frac{D_{RS}}{2}(F_{RS} - F_{RQ})\right]$$

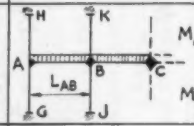
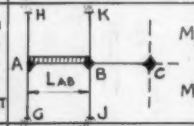
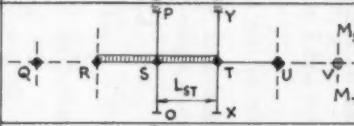
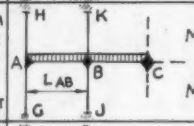
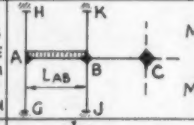
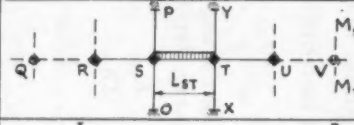
$$M_{TS} = -F_{TS} + D_{TS}(F_{TS} - F_{TU}) + \frac{D_{ST}}{2}(F_{SR} - F_{ST}) - D_{TS}\left[\frac{D_{ST}}{2}(F_{SR} - F_{ST}) + \frac{D_{UT}}{2}(F_{UT} - F_{UV})\right]$$

in which the factors F are the numerical values of the negative fixed-end bending moments at each end of each loaded span. The Code recommends that the remote ends of members continuous with the span being considered should be assumed to be fixed, an assumption which simplifies the formulæ. Therefore,

TABLE IV.

CONTINUOUS BEAMS
SUPPORT BENDING MOMENTS

BEAMS MONOLITHIC WITH SUPPORTS

CONDITIONS		END SPAN AB	INTERIOR SPAN ST
DEAD LOAD	ALL SPANS LOADED	 $M_{AB} = -\left[F_{AB} - \frac{D_{BA}}{2}\left(F_{BC} - F_{BA}\right)\right](1 - D_{AB})$ $M_{BA} = -\left(F_{BA} + \frac{D_{AB}}{2}F_{AB}\right)(1 - D_{BA}) - D_{BA}F_{BC}$	$M_{ST} = -\left[F_{ST} - \frac{D_{TS}}{2}\left(F_{TU} - F_{TS}\right)\right](1 - D_{ST}) - D_{ST}F_{SR}$ $M_{TS} = -\left[F_{TS} - \frac{D_{ST}}{2}\left(F_{SR} - F_{ST}\right)\right](1 - D_{TS}) - D_{TS}F_{TU}$
LIVE LOAD	MAXIMUM BENDING MOMENT AT L.H. SUPPORT	 $M_{AB} = -\left(F_{AB} + \frac{D_{BA}}{2}F_{BA}\right)(1 - D_{AB})$ $M_{BA} = -\left(F_{BA} + \frac{D_{AB}}{2}F_{AB}\right)(1 - D_{BA})$	 $M_{ST} = -\left(F_{ST} + \frac{D_{TS}}{2}F_{TS}\right)(1 - D_{ST}) - D_{ST}F_{SR}$ $M_{TS} = -\left[F_{TS} - \frac{D_{ST}}{2}\left(F_{SR} - F_{ST}\right)\right](1 - D_{TS})$
	MAXIMUM BENDING MOMENT AT R.H. SUPPORT	 M_{AB}, M_{BA} } FORMULÆ AS FOR "ALL SPANS LOADED"	$M_{ST} = -\left[F_{ST} - \frac{D_{TS}}{2}\left(F_{TU} - F_{TS}\right)\right](1 - D_{ST})$ $M_{TS} = -\left(F_{TS} + \frac{D_{ST}}{2}F_{ST}\right)(1 - D_{TS}) - D_{TS}F_{TU}$
	B.MS. AT SUPPORTS TO PRODUCE MAXIMUM POSITIVE B.M. ON SPAN	 M_{AB}, M_{BA} } FORMULÆ AS FOR "MAX. B.M. AT L.H. SUPPORT"	 $M_{ST} = -\left(F_{ST} + \frac{D_{TS}}{2}F_{TS}\right)(1 - D_{ST})$ $M_{TS} = -\left(F_{TS} + \frac{D_{ST}}{2}F_{ST}\right)(1 - D_{TS})$
STIFFNESS FACTORS (K) AND DISTRIBUTION FACTORS (D)		$K_{AB} = \frac{I_{AB}}{L_{AB}}$ $K_{AG} = \frac{I_{AG}}{L_{AG}}$ $K_{AH} = \frac{I_{AH}}{L_{AH}}$ $K_{BC} = \frac{I_{BC}}{L_{BC}}$ $K_{BJ} = \frac{I_{BJ}}{L_{BJ}}$ $K_{BK} = \frac{I_{BK}}{L_{BK}}$ $D_{AB} = \frac{K_{AB}}{K_{AB} + K_{AG} + K_{AH}}$ $D_{BA} = \frac{K_{AB}}{K_{AB} + K_{BC} + K_{BJ} + K_{BK}}$	$K_{RS} = \frac{I_{RS}}{L_{RS}}$ $K_{ST} = \frac{I_{ST}}{L_{ST}}$ $K_{SO} = \frac{I_{SO}}{L_{SO}}$ $K_{SP} = \frac{I_{SP}}{L_{SP}}$ $K_{TU} = \frac{I_{TU}}{L_{TU}}$ $K_{TX} = \frac{I_{TX}}{L_{TX}}$ $K_{TY} = \frac{I_{TY}}{L_{TY}}$ $D_{ST} = \frac{K_{ST}}{K_{RS} + K_{ST} + K_{SO} + K_{SP}}$ $D_{TS} = \frac{K_{ST}}{K_{ST} + K_{TU} + K_{TX} + K_{TY}}$
NOTATION		$F_{AB, \text{ETC.}}$ = NUMERICAL VALUE OF FIXED-END B.M. (NEG.) AT A (ETC.) DUE TO LOAD ON AB (ETC.)	$L_{AB, \text{ETC.}}$ = LENGTH OF MEMBER AB, ETC. $I_{AB, \text{ETC.}}$ = MOMENT OF INERTIA OF MEMBER AB, ETC.

as regards span ST, the ends of the beams at R and U, the ends of the lower columns at O and X, and the ends of the upper columns at P and Y are assumed to be fixed; that is D_{RS} and $D_{UT} = 0$. Substitution in the foregoing formulæ for M_{ST} and M_{TS} and reduction, gives the general formulæ

$$\left.\begin{aligned} M_{ST} &= -F_{ST} + \frac{D_{TS}}{2}(1 - D_{ST})(F_{TU} - F_{TS}) + D_{ST}(F_{ST} - F_{SR}) \\ M_{TS} &= -F_{TS} + \frac{D_{ST}}{2}(1 - D_{TS})(F_{SR} - F_{ST}) + D_{TS}(F_{TS} - F_{TU}) \end{aligned}\right\} \quad . \quad (1)$$

INTERIOR SPANS.—If span ST is an interior span in a frame the bending moments at critical sections are based on the conditions of loading given in the following.

Formulæ (1) are for cases when all spans are loaded, and therefore apply to the bending moments due to dead load on an interior span. To produce the greatest negative bending moments at support S due to live load it is necessary to omit the load from span TU. Similarly, for support T, the live load on span RS is omitted. To produce the maximum positive bending moment on span ST due to live load, it is necessary to load span ST only. The formulæ for these conditions of live loading are therefore variants of formulæ (1) and are given in *Table* IV with formulæ (1) written in a more convenient form.

END SPANS.—Consider the end span AB of the frame in *Table* IV. The formulæ for any span ST can be re-written to apply to an end span AB by substituting A, B, C, etc., for S, T, U, etc.; A is the end support, and there is no span corresponding to RS. The corresponding stiffness factors are: For beam AB, $K_{AB} = \frac{I_{AB}}{L_{AB}}$; for column AG, $K_{AG} = \frac{I_{AG}}{L_{AG}}$, etc., as given in the table. Similarly the distribution factors are

$$\text{At A.—} D_{AG} = \frac{K_{AG}}{\Sigma K_A}; \quad D_{AH} = \frac{K_{AH}}{\Sigma K_A}; \quad D_{AB} = \frac{K_{AB}}{\Sigma K_A},$$

in which $$\Sigma K_A = K_{AG} + K_{AH} + K_{AB}.$$

$$\text{At B.—} D_{BA} = \frac{K_{AB}}{\Sigma K_B}; \quad D_{BJ} = \frac{K_{BJ}}{\Sigma K_B}; \quad D_{BK} = \frac{K_{BK}}{\Sigma K_B}; \quad D_{BC} = \frac{K_{BC}}{\Sigma K_B}$$

in which $$\Sigma K_B = K_{AB} + K_{BJ} + K_{BK} + K_{BC}.$$

The general formulæ for the bending moments at supports A and B are therefore

$$\left.\begin{aligned} M_{AB} &= -\left[F_{AB} - \frac{D_{BA}}{2}(F_{CB} - F_{BA})\right](1 - D_{AB}) \\ M_{BA} &= -\left(F_{BA} + \frac{D_{AB}}{2}F_{AB}\right)(1 - D_{BA}) - D_{BA}F_{BC} \end{aligned}\right\} \quad . \quad . \quad (1a)$$

These formulæ are given in *Table* IV, in which also are given the formulæ for the maximum bending moments at these supports due to dead and live load and the corresponding formulæ for the bending moments at the supports for the

incidence of live load giving the maximum positive bending moment on span AB.

When the bending moments at the supports are known the positive and negative bending moments in the spans are obtained by combining the diagram of the free bending moments due to the load with the diagram of the support bending moments.

A Prestressed Suspension Bridge.

In the course of a lecture given in London at a meeting held under the auspices of the Cement and Concrete Association, Professor D. Vandepitte, of the University of Ghent, described a new type of prestressed suspension bridge designed by him.

The lecturer stated that the use of external cables had the disadvantage that the absence of bond reduced the load-carrying capacity of a beam, although this was largely off-set by the ease of construction and of maintenance and inspection. Prestressed beams continuous over three spans must frequently be deeper than necessary to accommodate the cables. By installing the cables below the beam and using vertical struts between the cables and the beams a smaller prestressing force, and hence a cheaper cable, was possible, but the cables would have to be well-braced and the height of the deck of the bridge would have to be increased to allow for the cables below.

In the new design the cables are placed above the bridge over the piers, and vertical hangers are used instead of struts. In this way suspended prestressed beams are formed which, as the span increases, have a large eccentricity; also the moments on the beams due to live load are smaller than those in an ordinary prestressed beam.

The bridge has a central suspended span of 183 ft. and two end spans of 59 ft. Compared with ordinary prestressed concrete there was a saving of 8 per cent., and this would be considerably greater for spans up to 300 ft. The sag of the cable is 13 ft. 3 in., that is one-fourteenth the span. The piers are of reinforced concrete, and the abutments are large reinforced concrete boxes filled with sand heavy enough to resist the forces imposed upon them.

The bridge is 27 ft. wide and consists of two girders 5 ft. 3 in. deep throughout their length, and 21 ft. 4 in. centre to centre; the lower flange is 7 ft. 6 in. wide and the top flange is formed by the deck slab. The vertical hangers from the cables are embedded at 13 ft. centres in the ends of transverse reinforced concrete slabs between the beams. The superstructure is connected to one abutment by means of a Mesnager-type hinge, and to the other by a free-moving pin and eye-bar joint.

Each cable consists of 510 galvanised steel wires of 0·2 in. diameter which form a cylinder of 5 in. diameter. At the end anchorages the cables are separated into three strands of 170 wires each. The entire superstructure was prestressed by pushing up the four intermediate support columns from the deck slabs by means of two jacks under each column. The raising of the columns elongated the cables 14¼ in., and produced the required tension of 1,570,000 lb. in each cable. Cast steel hinges were inserted between the columns and the deck slab before the jacks were removed. The cables are protected against corrosion by a tightly-wound layer of soft galvanised wire of 0·14 in. diameter.

The central span was tested with a uniform load of 545,000 lb., when the deflection was 1·42 in., which increased by 0·04 in. after one hour. A live load comprising four lorries with a total weight of 226,000 lb. was then applied, and the additional deflection was 0·63 in. When the test loads were removed there was no permanent deflection.

Water Tanks and Silos in Prestressed Concrete.

SEVERAL prestressed tanks and silos recently designed by M. C. Ostenfeld and M. E. Kalhague are described in the French journal "Travaux" for October and November, 1956. The authors state that in their experience it is preferable to have the cables in the thickness of the wall rather than outside the wall and covered with a protective layer of concrete. They also believe that it is preferable for the wall and floor to be monolithic rather than to have a so-called sliding joint at the base of the wall, as it is difficult to predict the degree of restraint that actually occurs at the joint. In many cases it has been possible to use mild steel reinforcement at the joint, and vertically in the walls instead of vertical prestressing cables. Many of the water towers consist of a thin wall of prestressed concrete supported on a thick reinforced concrete slab supported on columns, for which the allowable stresses may be those normally used in reinforced concrete buildings rather than the low stresses commonly permitted in water-retaining structures. Sliding shutters are also advised in order to avoid horizontal joints. In the following are described some of the sixty containers designed by the authors since 1947.

Circular Reservoirs.

Six tanks to contain fuel-oil, each with a capacity of 5300 cu. ft., were built in Denmark in the years 1947 and 1948. Each is 36 ft. diameter by 17 ft. high, with a wall 6¾ in. thick; the roof is a dome 5 in. thick (*Fig.* 1). Alternate rows of horizontal cables are anchored as shown in *Fig.* 2; the walls are also prestressed vertically.

Two water tanks, each with a capacity of 1,150,000 gallons, were constructed in 1952 at Stockholm. They are each 142 ft.

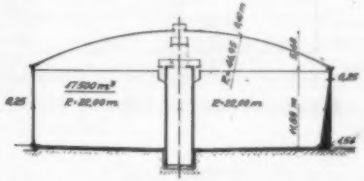


Fig. 3.—Comparison between Prestressed Tank (left) and Reinforced Tank (right).

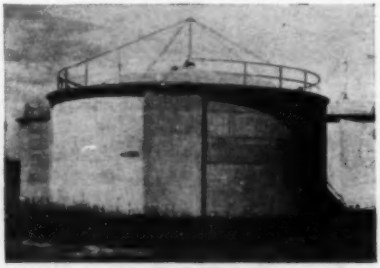


Fig. 1.—Tanks for Fuel-oil.

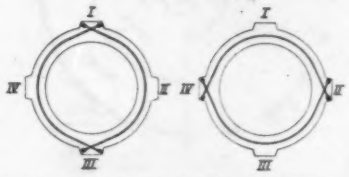


Fig. 2.—Method of Anchoring Alternate Rows of Circumferential Cables.

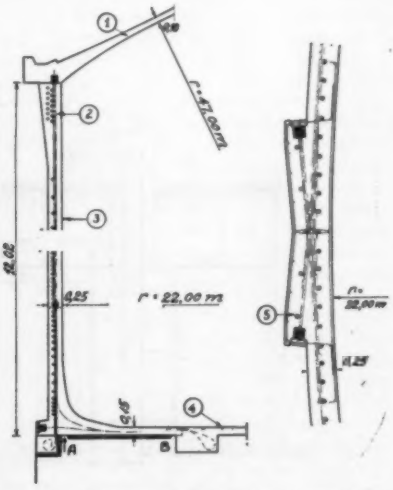


Fig. 4.—Cables in Tank of 1,150,000 Gallons Capacity.

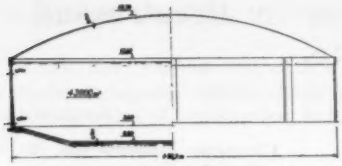


Fig. 5.—Tank of 2,820,000 Gallons Capacity.

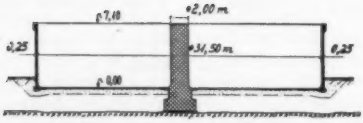


Fig. 6.—Tank for Cement Slurry.

Fig. 8.—Water Tower in Denmark.

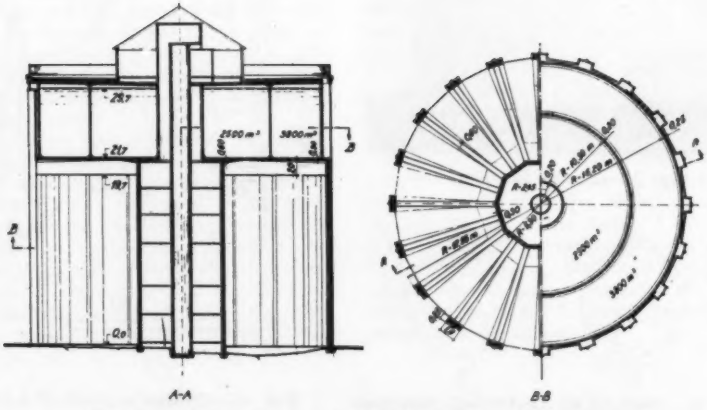


Fig. 7.—Water Tower of 415,000 Gallons Capacity.

Fig. 9*a*.—Water Tower in Denmark.

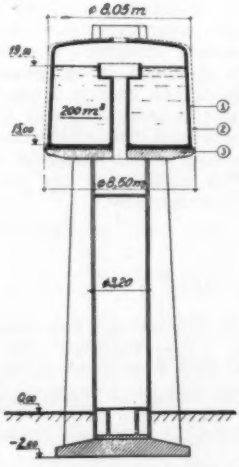


Fig. 9*b*.—Section through Tower shown in Fig. 9*a*.

diameter by 38 ft. high, with walls 9¾ in. thick. *Fig.* 3 is a cross section through one of the tanks showing on the left the dimensions of the prestressed tank and on the right the dimensions of an alternative design in reinforced concrete for which the maximum allowable tensile stress was 215 lb. per square inch. The arrangement of the cables in a wall is shown in *Fig.* 4.

A circular reservoir of 2,820,000 gallons capacity has been built at Oakland, California (*Fig.* 5); this has a diameter of 202 ft. and the wall is 40 ft. high. To reduce the height of the wall the base is saucer-shaped, and the thickness of the wall varies from 2 ft. at the base to 1 ft. at the top. Horizontal and vertical cables are used, and in this case there is a free joint between wall and base.

Two tanks for cement slurry (*Fig.* 6) were built in Denmark. They are 71 ft. diameter and 16 ft. high. The central towers supporting the agitator arms are of masonry, and pass through the base with a flexible joint between the base and the tower. The wall and base are monolithic in both tanks.

Water Towers.

A water tower at Vasteras, Sweden, has a total capacity of 415,000 gallons in two annular compartments (*Fig.* 7). In this case the bottom of the tank (1 ft. thick) is supported by twenty radial beams carried on columns externally and on a central tower internally. The bottom of the tank is 70 ft. above ground, and the columns are not braced in this height. The external and intermediate walls are prestressed vertically and horizontally. The radial beams and the floor are also prestressed, the latter by two circumferential ties each comprising ten cables of twelve 0·2-in. wires. The beams were cast first and partially prestressed to support the weight of the floor, which was cast monolithically with the walls.

The water towers shown in *Figs.* 8 and 9 both comprise prestressed tanks with thin walls supported on elevated platforms of reinforced concrete. That at Aarhus, Denmark, has a capacity of 53,000 gallons (*Fig.* 8). The tank has a diameter of 40 ft. and a height of 25 ft.; the thickness of the wall is 6 in. and the base is 4 in. thick. The base is cast on a layer of bituminous material allowing movement of the slab. The roof is a conical dome 4 in. thick.

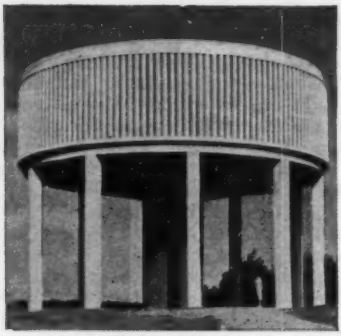


Fig. 10.—Water Tower in Denmark.

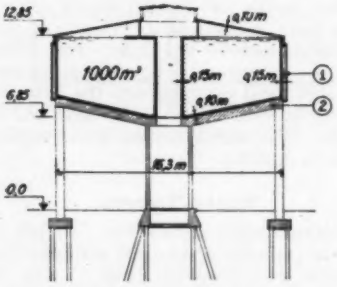


Fig. 11.—Section through Tank shown in Fig. 10.

The wall is prestressed horizontally only, the circumferential prestressing being used also to prestress the dome and base. *Figs. 9a* and *9b* show a tank of 13,000 gall. capacity at Haderslev, Denmark, constructed in a similar manner. A tank at Bronderslev, Denmark (*Figs.* 10 and 11), has a capacity of 65,000 gallons. Built in a similar manner to those previously described, the thickness of the external wall was reduced by making the bottom of the tank conical.

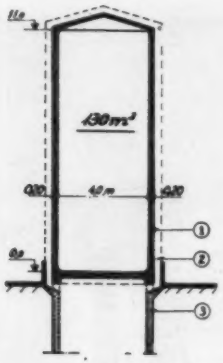


Fig. 12.—Tank for Hot Water.

A tank at Boden, Sweden, has a capacity of 8000 gallons; it is 35 ft. high and 13 ft. diameter, with a wall 8 in. thick (*Fig.* 12). It is designed to contain water varying in temperature from 90 deg. to 5 deg. C. The wall is prestressed circumferentially and vertically. Only 10 per cent. of the prestressing force is required to resist hydrostatic pressure, the remainder being provided to resist the forces caused by changes in temperature of the water.

Silos.

Several cylindrical silos to contain cement and sugar have been constructed to the authors' designs. The capacities vary from 35,000 cu. ft. to 270,000 cu. ft. The walls of the cement silos are prestressed circumferentially and vertically because of temperature stresses caused by filling the silos with hot cement from the mills. The sugar silos are prestressed circumferentially only.

Quick Construction.

THE reinforced concrete frame for an apartment house in New York City was completed in 51 days. The structure is 14 stories high, covers a floor area of 8000 sq. ft., and contains 280 rooms. Most of the stories above the third were built at the rate of three days each. The floors are of the flat-plate type 6½ in. thick, and each floor contains 230 cu. yd. of concrete with an average of 5½ lb. of steel per square foot. The concrete was mostly placed by crane and bucket.

Design Based on the Stress in the Reinforcement.

By J. S. SAVONA, A.M.I.Struct.E.

SINGLY-REINFORCED members subjected to bending are usually designed on the criterion of the maximum stress in the concrete. If the strength of the concrete is more than sufficient to resist the compressive stresses produced by the steel at working load the following method may be used.

(1) Determine the bending moment M. (2) Determine $Q = \frac{M}{bd^2}$.

(3) Because $t = \frac{Q}{a_1 p}$, $a_1 = \frac{Q}{tp}$.

(4) Substitute $\frac{Qm}{t} = \frac{n_1^2\left(1 - \frac{n_1}{3}\right)}{2(1 - n_1)} = f(n_1)$.

Proof.—Since $\frac{Q}{t} = a_1 p$, $p = \frac{n_1^2}{2m(1 - n_1)}$, and $a_1 = 1 - \frac{n_1}{3}$.

Therefore $\frac{Q}{t} = \frac{n_1^2\left(1 - \frac{n_1}{3}\right)}{2m(1 - n_1)}$.

Multiplying by m, $\frac{Qm}{t} = \frac{n_1^2\left(1 - \frac{n_1}{3}\right)}{2(1 - n_1)}$.

Plotting $\frac{Qm}{t}$ as ordinates against n_1, with corresponding values of a_1 as abscissæ, the curve in *Fig.* 1 is obtained; the other curve is obvious.

EXAMPLE I.—A slab is subjected to a bending moment of 95,000 in.-lb., the depth to the steel is $7\frac{1}{2}$ in. and can resist a stress of 25,000 lb. per square inch. Determine the area of reinforcement required. Assume a modular ratio of 12.

$$Q = \frac{95{,}000}{12 \times 7{\cdot}5^2} = 141. \quad \frac{Qm}{t} = \frac{141 \times 12}{25{,}000} = 0{\cdot}0678. \quad \frac{Q}{t} = \frac{141}{25{,}000} = 0{\cdot}00564.$$

Interpolating from *Fig.* 1:

$$n_1 = 0{\cdot}321. \qquad a_1 = 0{\cdot}893. \qquad \frac{a_1 n_1}{2} = 0{\cdot}1435.$$

Therefore $p = \frac{Q}{ta_1} = \frac{0{\cdot}00564}{0{\cdot}893} = 0{\cdot}0063$. $A_s = 0{\cdot}0063 \times 12 \times 7\frac{1}{2} = 0{\cdot}566$ sq. in.

Say, two layers of mesh No. 105 (B.S. 1221A).

$$c = \frac{Q}{a_1 \frac{n_1}{2}} = \frac{141}{0{\cdot}1435} = 984 \text{ lb. per square inch.}$$

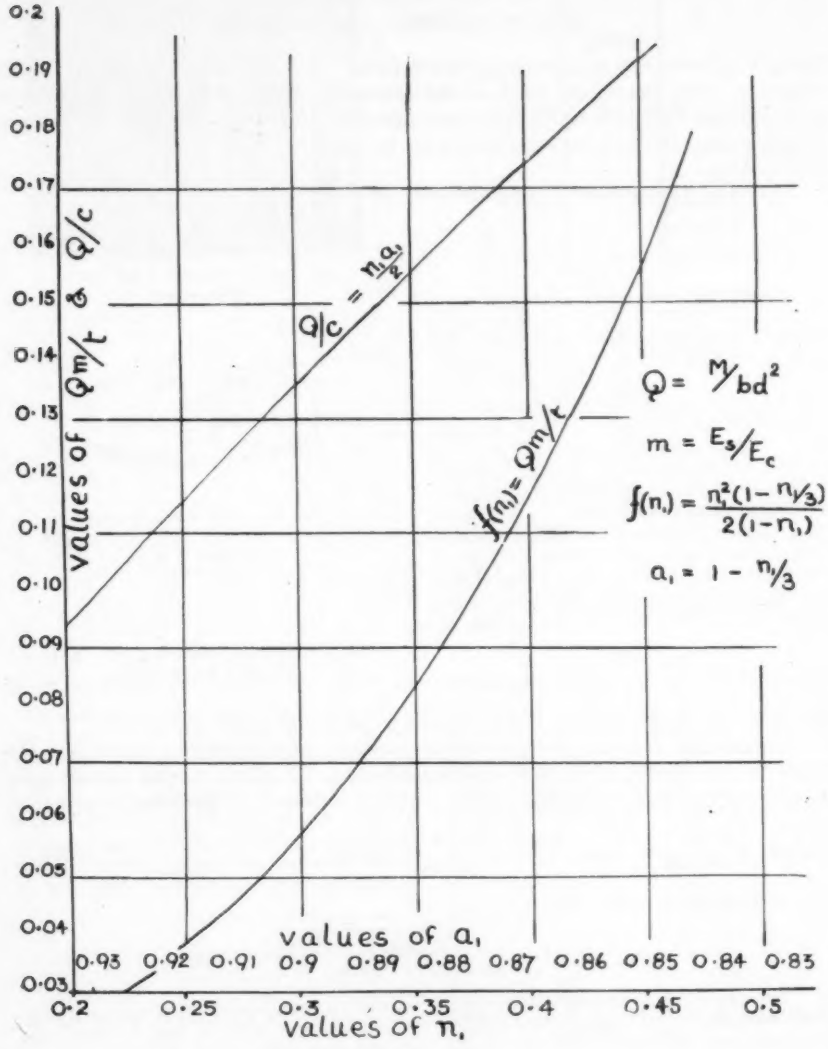

values of Qm/f & Q/c
0·2
0·19
0·18
0·17
0·16
0·15
0·14
0·13
0·12
0·11
0·10
0·09
0·08
0·07
0·06
0·05
0·04
0·03
Q/c = n₁a₁/2
f(n₁) = Qm/f
Q = M/bd²
m = Es/Ec
f(n₁) = n₁²(1 − n₁/3) / 2(1 − n₁)
a₁ = 1 − n₁/3
values of a₁
0·93 0·92 0·91 0·9 0·89 0·88 0·87 0·86 0·85 0·84 0·83
0·2 0·25 0·3 0·35 0·4 0·45 0·5
values of n₁

Fig. 1.

EXAMPLE II.—A beam 2 ft. 4 in. deep by 1 ft. 3 in. wide is subjected to a bending moment of 3,258,000 in.-lb. If the reinforcement can sustain a stress of 32,000 lb. per square inch, assuming m to be 15, determine the area of steel required if the concrete can resist a stress of 2000 lb. per square inch.

$$Q = \frac{3{,}258{,}000}{15 \times 28^2} = 278. \quad \frac{Qm}{t} = \frac{278 \times 15}{32{,}000} = 0{\cdot}131. \quad \frac{Q}{t} = 0{\cdot}00869.$$

By interpolation from *Fig.* 1, $n_1 = 0{\cdot}42$. $a_1 = 0{\cdot}86$. $\frac{a_1 n_1}{2} = 0{\cdot}181$.

Therefore $p = \frac{Q}{ta_1} = 0{\cdot}0101$. $A_t = pbd = 0{\cdot}0101 \times 15 \times 28 = 4{\cdot}34$ sq. in.

$$c = \frac{Q}{\frac{a_1 n_1}{2}} = \frac{278}{0{\cdot}181} = 1536 \text{ lb. per square inch.}$$

Requirements for Fire-resistance.

THE following is abstracted from "Fire Research 1956" (the annual report of the Fire Research Board of the Department of Scientific and Industrial Research, published by H.M.S.O. at 4*s*.).

While the changes in the last revisions of the building by-laws in England and Scotland are an advance towards a more liberal form of building control, the performance standards may be restrictive if they are set too high. Investigations have therefore been started to obtain information that will enable a more "scientific appraisal" of the risks to be made for any further revision of the by-laws.

EXTERIOR WALLS, AND MOVEMENT OF SMOKE IN HIGH BUILDINGS.—The requirements for fire-resistance and non-combustibility in external walls have been considered to be unduly restrictive. Arrangements have therefore been made for full-size tests on a four-story tower which will also be used to investigate the movement of smoke and hot gases in staircases of high buildings, and to establish requirements for ventilation and venting. Similar work is being planned on smaller scales.

ROOFS.—There is no test to which by-laws can refer in requiring roofs to afford protection against external fire. The fire-resistance tests of B.S. 476 are not suitable for this purpose, and so far it has not been possible to devise a test suitable for grading roofs. A fire test for roofs is now being discussed by a committee of the British Standards Institution, and tests have been made on bituminous felts on concrete. All the organic-based felts examined permitted the spread of flame, the distance increasing with the number of layers of felt. There was no spread of flame in the case of asbestos-based bituminous felts on concrete. In tests of felts on wooden decks, although there was spread of flame on asbestos-based bituminous felts it was less than on those with organic bases. It would appear that the base material of bituminous felts and the material of the decking on which the felt rests can both influence the spread of flame.

A Bridge in Venezuela.

A CONTRACT has been placed for the construction of a prestressed concrete bridge over the entrance to Lake Maracaibo, Venezuela. The bridge will be more than six miles long, and there will be a movable part to allow ships to pass. The successful tenderers are Precomprimido, C.A., of Caracas, and Julius Berger, A.G., of Germany. The contract price is £34,000,000.

Method of Forming Joints in Roads.

An arrangement devised for the purpose of improving the riding quality over expansion joints in concrete roads and of providing a method of making a complete expansion joint including the placing of dowel-bars, with a saving of time and labour, is shown in *Fig.* 1.

A row of chairs is placed on each side of the joint before the concrete is laid and these support dowel-bars which pass through a compressible joint filler. Provision is made for the dowel-bars to move within the concrete, for placing the chairs to the vertical bars of the chairs, beyond which they extend on both sides. They are coated with a bituminous preparation over half their length to prevent this part from bonding with the concrete, and a cap, containing a compressible filling to allow movement of the dowel, is fitted on the coated end of each bar.

The concrete is laid continuously over the joint-filler. When the concrete has hardened sufficiently a power-driven saw is used to cut it away over the filler, and the joint-sealer is placed in the channel

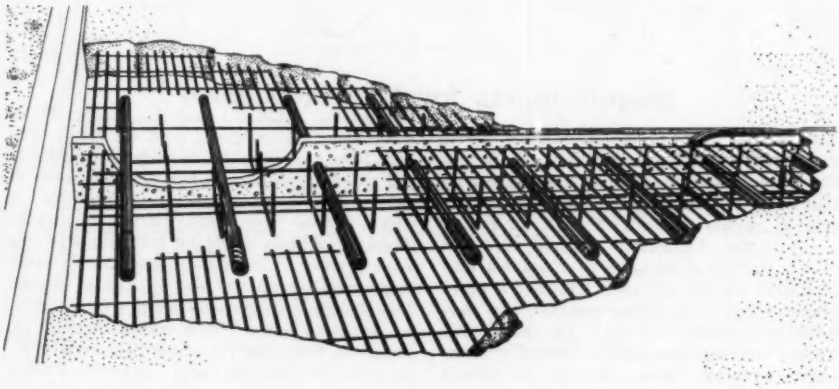


Fig. 1.

at the required distance from the joint, and for placing a joint-sealer on the filler. The size of the chairs and the number of the dowel-bars can be varied as required.

The chairs are made of welded steel rods and when in position form a trough-like structure resting on the formation. Rods are welded on the chairs, one to the base and others, at equal heights from the base, to the vertical parts. The chairs are secured by pickets spiked into the formation.

The chairs are placed with their longer sides parallel to the joint, and extend the full width of the slab. The joint filler is of suitable compressible material, and is drilled to receive the dowel-bars. Distance-pieces can be fixed between the chairs and the filler to keep the filler in position. The dowel-bars are fastened thus formed. A plastic twin-tube can be used for this purpose; this tube limits the extrusion of the sealing material under compression when the concrete expands due to a rise of temperature and returns to its normal shape when the concrete contracts. The diameter of the tube is normally $\frac{1}{4}$ in. greater than the width of the joint, and is reduced by extracting the air by means of a vacuum pump; after placing the tube in position the air is allowed to return to the tube. This restores its shape and forms a seal between the faces of the concrete joint. It is claimed that the plastic tube provides a watertight seal, is unaffected by fuel oil, and can be made in the same colour as the concrete. The method is patented by the British Reinforced Concrete Engineering Co., Ltd.

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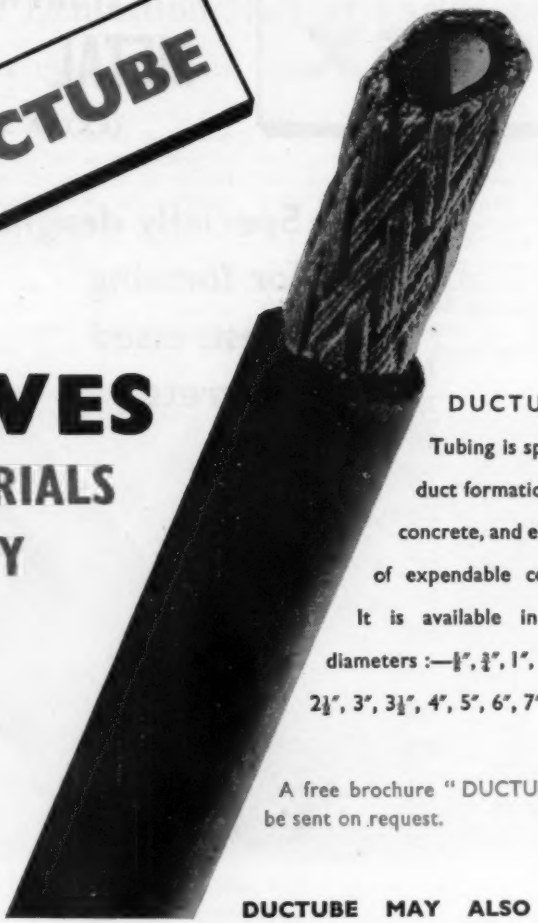

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It is available in 10 ft. lengths with screw or bell ends, with internal diameters of ½″ to 2½″. Each length is simply threaded on the cable and screwed or butted into the preceding length. The joins are covered with tape for added protection against the concrete grout entering the duct. The whole cable encased in steel is then placed in the structure in the design position.

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AN UNUSUAL PRESTRESSED SHELL ROOF.

COVERING an area of 3800 sq. yd., the roof shown in *Fig.* 1 is stated to be the first of this type to be constructed in Europe. On plan the roof is an ellipse with a major axis of 240 ft. and a minor axis of 160 ft.; around the periphery is a tie-beam supported on thirty-six columns of varying height so that the upper surface of the beam follows the shape of a cylinder concave upwards. The columns are spaced equally around the beam at about 18-ft. centres. The roof proper has the general shape of a saddle; along the major axis of the ellipse it has the form of a catenary rising 14 ft. 6 in. from the centre to the edges but transversely across the minor axis it is a very flat arch with a rise of 4 ft. It is of concrete 2¼ in. thick prestressed in two directions by 1-in. diameter high-tensile bars arranged as shown in *Fig.* 2. A description of the structure is given in the Belgian journal "La Technique des Travaux", from which the following is abstracted.

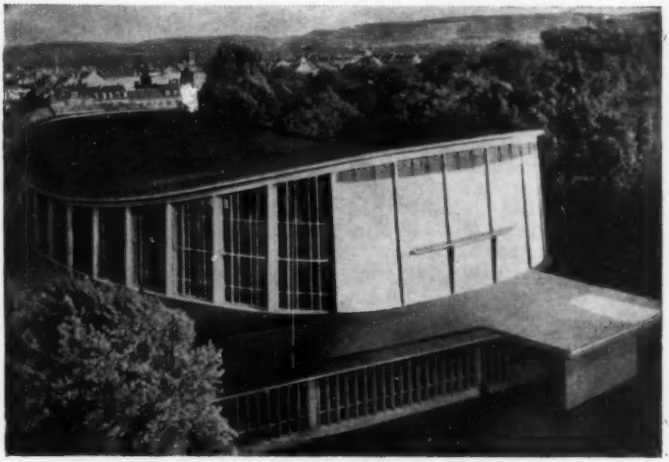


Fig. 1.

For the purpose of design the dead and imposed loads were assumed to be 41 lb. and 15 lb. per square foot respectively, producing membrane forces in the longitudinal direction of 10·4 tons per linear foot due to the dead loads and 4 tons per linear foot due to imposed loads; transversely the membrane forces were 3·4 tons and 1·2 tons per linear foot respectively. Under the total loads there is a resultant longitudinal tensile force of 2000 tons in the slab and a corresponding compressive force of 1000 tons in the tie-beam. These forces are considerable for such a thin slab and do not allow for bending moments of any magnitude to be resisted; therefore, the shape of the roof was determined so that the dead loads would not cause bending moments while those due to imposed loads were reduced by the prestressing forces. The stresses were calculated by the theory of thin shells, the applicability of which was shown by the absence of cracks and the behaviour of the structure.

It was necessary to shutter the entire roof at once and this was done entirely in timber. The soffit of the thin slab was left as cast against the shutters, and a

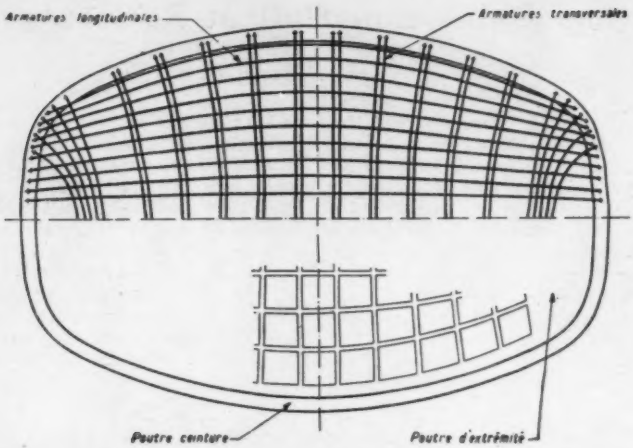


Fig. 2.—Arrangement of Prestressing Bars.

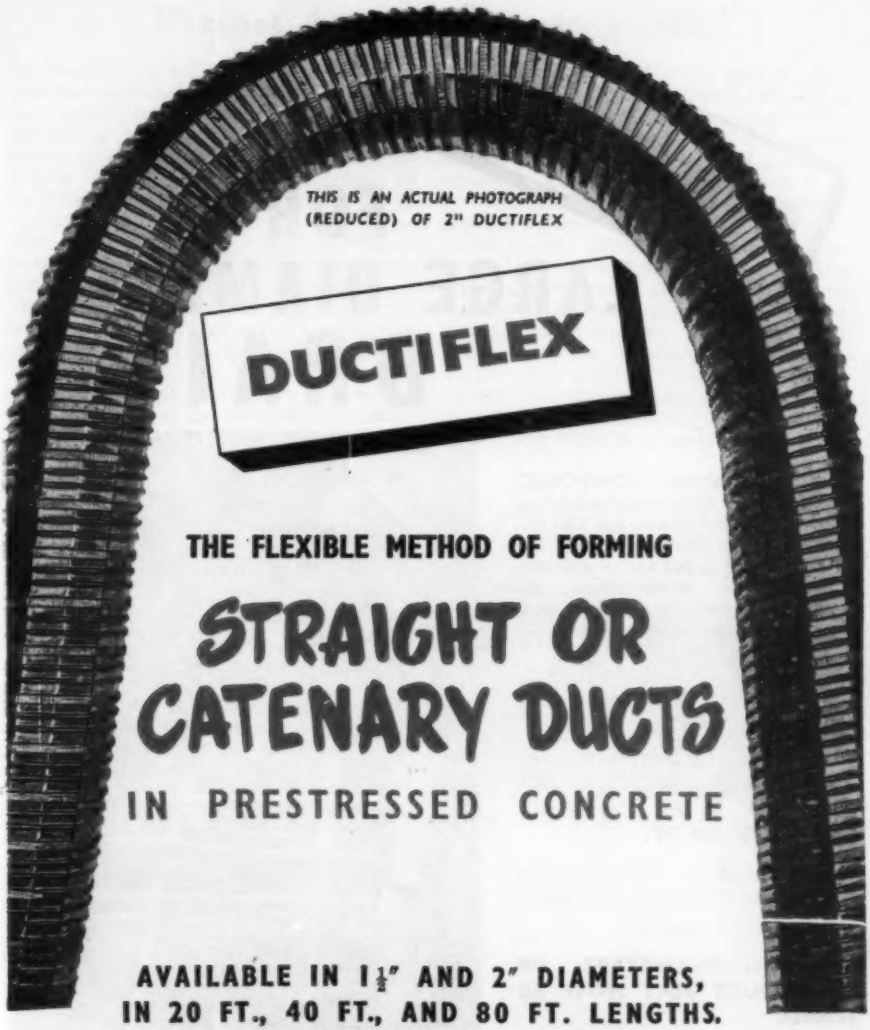
patterned surface (see pages 55 and 56 of this journal for January 1956) was obtained by the arrangement of the longitudinal and transverse timber bearers which were set below the general level of the soffit. The concrete had a crushing strength at 28 days of 5112 lb. per square inch, the water-cement ratio was 0·46, and both internal and external vibration was used. The entire work, including design, was carried out in five months.

The architect is Professor E. Schelling and the engineer is Dr. U. Finsterwalder, chief engineer of Messrs. Dyckerhoff & Widmann who collaborated in the construction with Messrs. Wayss & Freytag.

Research in France.

THE Association Française de Recherches et d'Essais sur les Matériaux de Construction has started publication of technical and scientific studies in the form of small booklets which will be available from Documentation Technique du Bâtiment et des Travaux Publics, 6 rue Paul Valéry, Paris XVI. The first study, entitled "Boussinesq's Problem", is by Professor Hjalmar Granholm of the Chalmers Technical High School, Sweden; it comprises 24 pages, and is priced at 300 francs. The second is entitled "Studies of the Shrinkage of Concrete in the North African Climate" (price 450 francs) by J. Delarue and R. Berthier of the Laboratoire Public d'Essais et d'Études of Casablanca.

Forthcoming brochures in this series include "Bearing Capacity of a Solid Composed of Two Different Plastic Layers", by Y. Tcheng of the Faculté de Paris; "Précis of Statistics applied to the Testing of Materials", by R. L'Hermite of the Centre Experimental du Bâtiment et des Travaux Publics; "The Movements of Water in Soils under the Influence of Temperature", by P. Habib of the Faculté de Paris and F. Soeiro of the Higher Technical Institute, Lisbon; and "Antiseismic Structures", by A. Roussopoulos of the Polytechnic, Athens.



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Precast Arch Bridges in Spain.

In the fifteenth volume of the Memoirs of the International Association for Bridge and Structural Engineering, Sr. C. Fernández Casado describes the methods now being used by the Spanish Ministry of Public Works for the construction of bridges using precast arches. Precast construction is used because of the reduction in the scaffolding, which is particularly important in the case of a bridge over a deep valley; other advantages include a reduction of the time the bed of a river need be obstructed by staging, quicker construction, and smaller foundations because of the reduction in dead weight of the superstructure due to the higher quality of the concrete and consequently the smaller sizes of members.

The first bridge (*Fig.* 1) built in this manner is over a canal at the Cubillas dam in Granada. This is a three-pinned arch with solid spandrels and a road slab acting compositely to resist the longitudinal stresses. The spandrels are continued on to the embankments to form walls retaining the filling. The bridge has a span of about 150 ft. Precast arches and transverse beams form the first part of the structure on which the spandrels were cast, and this construction was used to support centering for the slab. The arches were cast in halves and erected by means of a mast and temporarily supported on a central timber tower. On the same site an aqueduct with eight spans of 66 ft. was built in a similar manner.

Fig. 1.—Bridge at Cubillas, Spain, with Span of 150 ft.

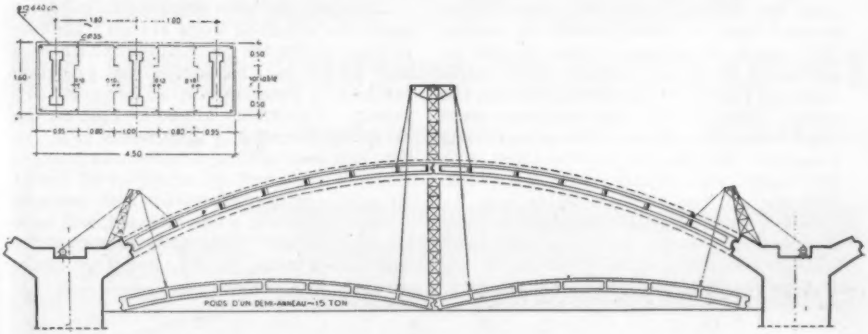

Fig. 2.—Erection of Bridge at Mérida.

The bridge at Mérida over the river Guadiana has eight spans of 198 ft. and a width of 60 ft. Each span comprises three precast arch beams cast in halves supporting prestressed precast cross-beams on which the slab was cast in situ. Each arch was formed in the following manner. Precast curved beams were floated on barges to the site where they were erected (*Fig.* 2) to form three-pinned arches resting on the abutments, which cantilever about 20 ft. to reduce the precast part of the span. The beams are of I-shape and are sufficiently strong to support the shuttering for the concrete cast around them to form rectangular arches. This work was done in four stages, as shown in *Fig.* 2. Initially the beams were pin-jointed at the abutments and the crown ; the joints at the abutments were fixed when the concreting of an arch was complete, but the joint at the crown was left until the bridge was completed. Erection of the curved beams was by hoists from a temporary steel tower on a concrete pier in the river bed and small towers on each abutment.

A two-pinned tied arch over the river Tinto at San Juan del Puerto has a span of about 330 ft. The arch is hollow and is formed by two precast I-shaped members joined by top and bottom slabs cast in situ. The I-shaped members were cast in sixteen parts and taken to the site by barge. Four of the parts were joined together to form a quarter-arch, which was erected from temporary steel towers in the bed of the river and on the abutments. Precast beams were used for the transverse members supporting the in-situ slab and for the wind-bracing between the arches.

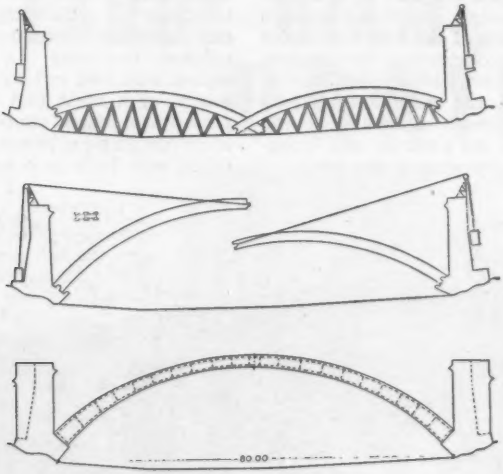

Fig. 3.—Method of Erecting Bridge of 250 ft. Span.

Designs have been prepared for a bridge spanning 250 ft. at Vega Terron ; the two precast arches are to be similar to those of the bridge at Mérida. The proposed method of construction is, however, different. The arches are to be cast on the ground between the abutments (*Fig.* 3). One end is to be hinged to the abutments ; the free ends will be attached to cables passing over the abutments and fastened to weights which, when released, will raise the half-arches. Final adjustments of the position of the arches is to be by hydraulic jacks at the crown.

Photographs of Tunnel Work in progress and Segment Moulds used

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PRECAST FRAMES.

THREE of the structures being erected on the Thames View housing estate at Barking are nine-story buildings, each containing 48 one-bedroom flats, with precast reinforced concrete frames. All the internal partitions are in hollow clay blocks. The site is reclaimed land beside the river, and the level was raised about 4 ft.; all the structures are therefore built on piled foundations. The structures are built with factory-made precast frames, stairs and landings, roofs and floors. Erection commenced in August last year and the first building was completed ten months later. It is expected that with the experience gained it will be possible to erect tall buildings at the rate of a floor every ten days.

In the first structure completed the average number of men employed during erection was 13, and a total of 15,143 man-hours was worked. In the second block only 9729 man-hours of labour were required. The quantities of precast concrete used in each block included 307 tons of columns, 217 tons of reinforced beams, 138 tons of prestressed beams, and 423 tons of floor slabs; in addition there are 149 tons of flooring cast in place.

The completed structure has a central part nine stories high and two wings of eight stories (*Figs.* 1 and 2). The central part has exposed concrete columns and brick panels on both elevations, and recessed balconies on one side. The concave faces of the end wings have

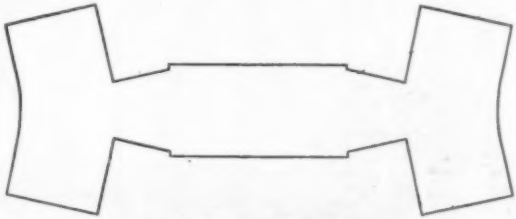

Fig. 1.—Outline Plan.

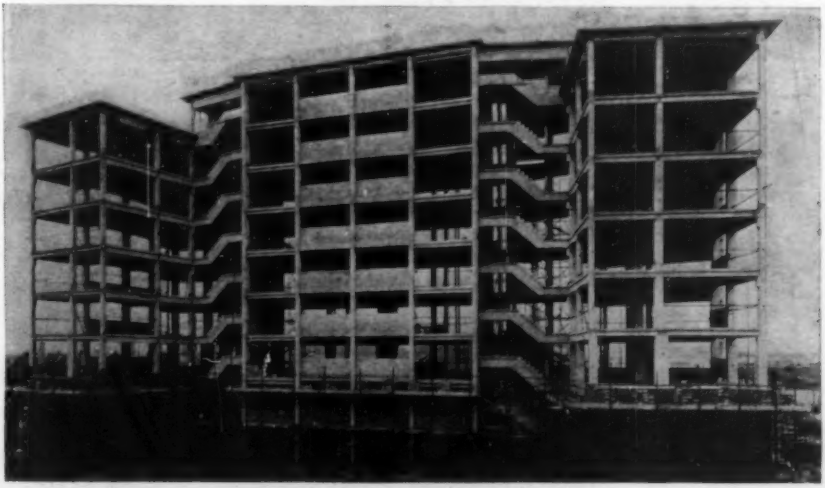

Fig. 2.—The Frame Completed.

exposed concrete columns and brick panels; all the other elevations are in brick.

Between the central part and each wing are a brick-covered lift shaft and staircase; structurally they are part of the central part, and there are expansion joints between them and the wings. The roof slab is thermally insulated from its supporting beams. The roof overhangs the frame, and suspended from it is a track for a travelling crane for use during construction and for maintenance work.

The exposed columns are 10 in. wide, and so that the joints at eye level will not be obvious the first lifts of concrete are as tall as possible and vary in height from column to column. These columns are joined as high above ground level as possible, and adjacent columns are not generally joined at the same level. This arrangement also simplified erection.

To provide resistance to the wind there are three reinforced concrete stiffening walls, 5 in. thick and cast in half-story heights, within the central part; the

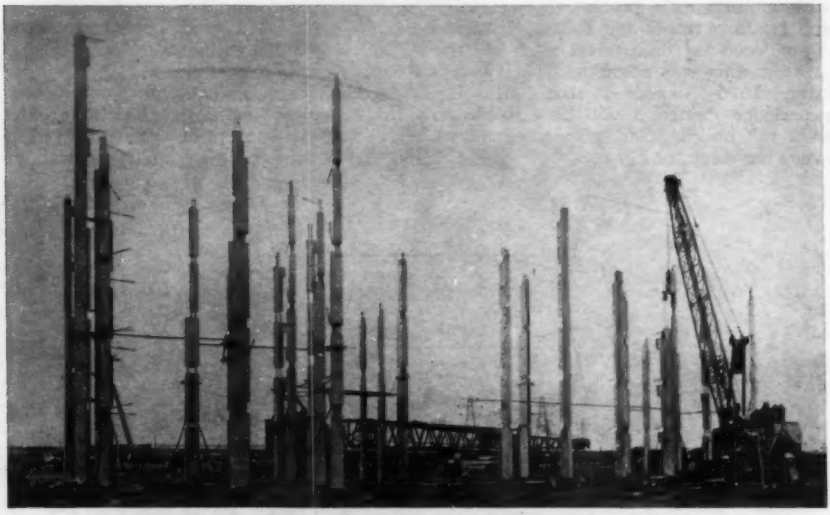

Fig. 3.—Some of the Precast Columns Erected.

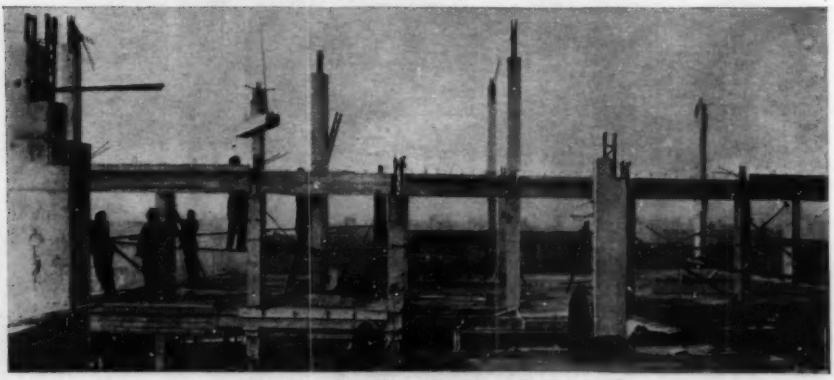

Fig. 4.—Erecting Precast Beams.

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parapet walls of the balconies were also cast in place to provide stiffness. The frame is designed as continuous for live loads and finishes and as simply supported for the weight of the precast structure. The beams were fixed without props. In some cases the sizes and proportions of the columns are governed by architectural considerations, being either deep in cross section or larger than would have been necessary if they had not to carry beams on a skew. Most of the columns are, however, designed only for structural requirements.

The edge-beams generally are of reinforced concrete and their main function is to carry the brickwork. The edge-beams along the skewed faces of the end wings carry the floors as well as the walls, and are of prestressed concrete.

The internal floor beams are of prestressed concrete ; they are rectangular in cross section and, to increase the structural depth, they have central upstands equal in height to the floor slabs (5 in.) extending along the central three-fifths of the beam. The floor beams extend transversely across the central part and the wings.

The completed building is nearly 80 ft. high at the central part and about 112 ft.

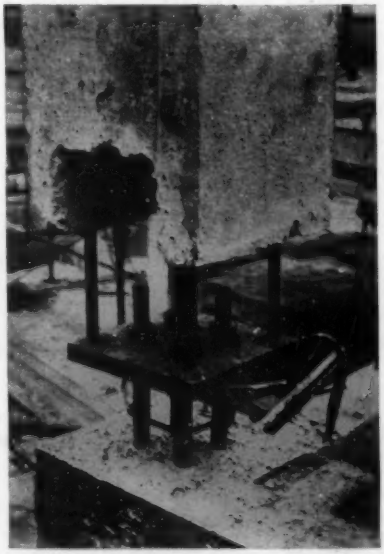


Fig. 5.—Joining the Columns.

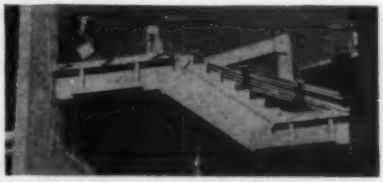


Fig. 6.—Precast Staircase.

long. The central part is 82 ft. long and its greatest width is 26 ft. 5 in. across the exposed columns. The end wings are about 71 ft. high, 57 ft. long, and 25 ft. wide. The edge-beams of the staircase bays are integral with the main frame of the central part, and are supported on the columns of the wings.

For reasons of economy and appearance the columns (*Figs.* 3 and 4) were cast as long as possible, and in this case their length was controlled by the cranes available. A mobile crane of 8 tons' capacity was used for fixing the first lifts of column, and a travelling tower crane of 30 cwt. capacity at maximum reach for the upper lifts. The maximum length of column lifted by the mobile crane was 5-stories, and the upper lifts were fixed for the additional stories.

Eighteen of the columns were cast in one piece to extend through five stories, 25 through four stories, 14 through three stories, and 139 through one, two, or three stories for use in the upper stories. The first joints of adjacent columns are at the fourth floor and fifth floor or at fourth floor and third floor ; most of the subsequent extensions are in two-story lengths. The columns to carry the beams are not designed to carry any load other than that due to erection.

The joint between the two parts of a column (*Fig.* 5) comprises a steel plate and threaded bars which fix the upper and lower parts of a column after vertical alignment. At each joint the upper part has a plate welded to four protruding bars. The lower part of the column has four protruding bars with threaded ends, which are protected from damage during handling by covering them with short lengths of hosepipe. One bar is provided with an aligning nut and a lock nut ; the other three have aligning nuts only. The lower part of the column is first erected, and the length of the upper part is next measured in order to ascertain the

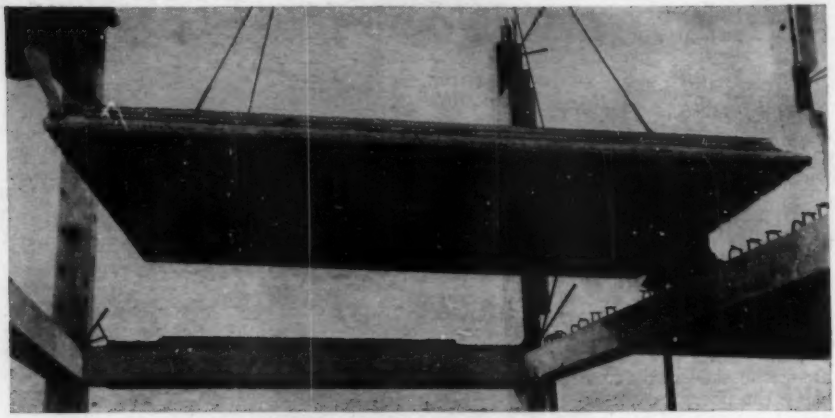


Fig. 7.—Erecting a Floor Slab.

required height to the underside of the plate, and the locked aligning nut is then adjusted to correspond with this dimension. The upper part of the column is then hoisted into position, its splice-plate is slid over the four bars projecting from the lower part of the column and rested on the locked aligning nut. The other nuts are then adjusted to plumb the upper part of the column, and nuts are screwed on the threads on top of the plate.

Each staircase (*Fig.* 6) has two precast flights per story, with a half-landing.

The precast slabs (*Fig.* 7) are made with hollow cores and are 5 in. deep for the greater part of the floors and roof, and are solid for the landings and the roof above them. Most of the floor and roof slabs are 7 ft. 6¼ in. wide, many being non-rectangular in shape and many having holes and cut-outs formed in them. The roof slabs have notched ends and are fixed together with steel tie-bars and concrete filling.

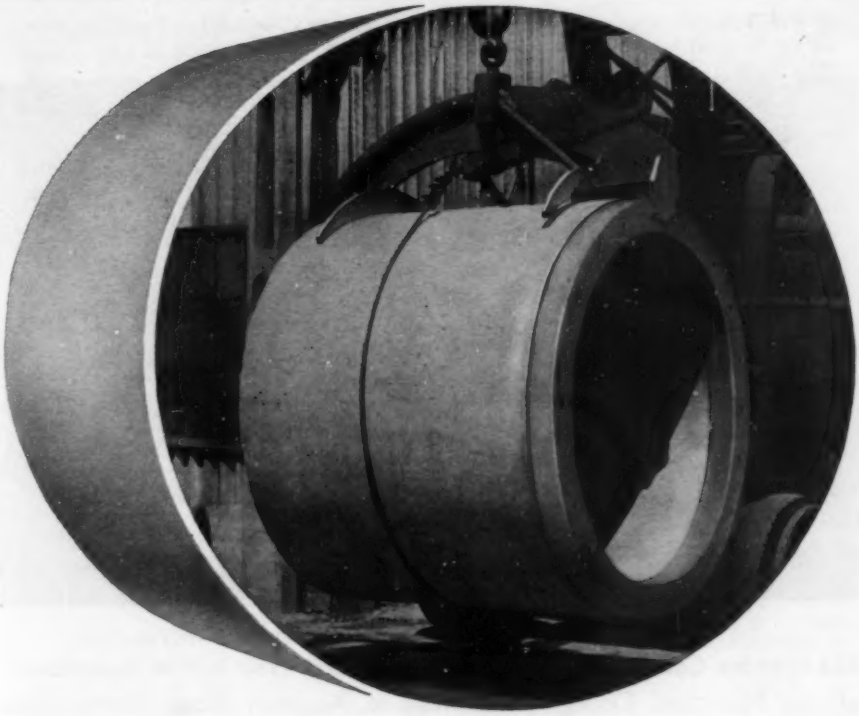
Precast Crane Track.

An innovation was the use of prestressed precast beams to carry the track for the tower crane. They were designed for a working load of 40 tons. They were made to be easily erected and removed, and are now in use on another contract. The beams were arranged to rest on pile caps at the ends of precast concrete tie-beams and were fixed in position by means of fishplates. The beams were in lengths of 17 ft. 9 in. and 18 ft. 9 in. by 1 ft. wide and 2 ft. 7 in. deep. The pile caps are 2 ft. square.

Mr. C. C. Shaw is the Borough Architect of Barking, the general contractors are Carlton (Contractors), Ltd., and the subcontractors for the precast work are Concrete, Ltd.

A New Film on Piling.

A NEW film; now available on loan from the Franki Compressed Pile Co., Ltd. (39 Victoria Street, London, S.W.1), illustrates the methods employed in the construction of Franki piles and shows various contracts carried out by the Company. The film was recently shown to delegates at the Fourth International Conference on Soil Mechanics and Foundation Engineering, and should be of interest to universities and technical colleges. It requires a 16-mm. sound projector.



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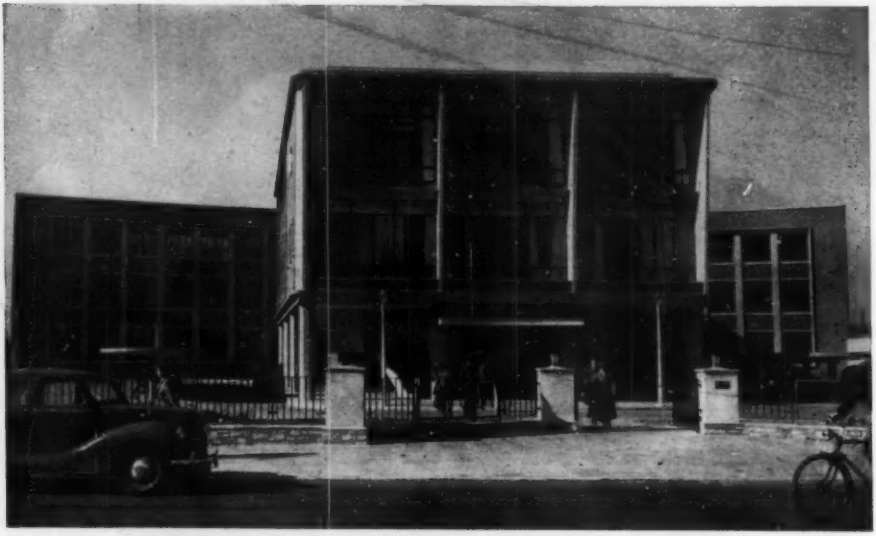

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At many colleges there are courses during the winter dealing with various aspects of concrete work. Details of the courses can be obtained from technical colleges throughout the country, or from the Cement and Concrete Association, 52 Grosvenor Gardens, London, S.W.1.

British Contracts Overseas.

ACCORDING to the Ministry of Works, during the year ended March 31, 1957, British building and civil engineering contractors carried out work oversea to the value of £94,000,000, an increase of £7,000,000 over the previous year. Nearly sixty firms were working in more than fifty countries. The value of new contracts obtained during the period was £92,000,000, compared with £109,000,000 in the previous year; this reduction is attributed to the political position in the Near and Middle East. It is reported that U.S. firms carried out work oversea to the value of £250,000,000 last year, and Italy and Germany are also competing keenly.

There have been considerable increases in the value of new work obtained in the West Indies, New Zealand, Nigeria, and Pakistan, but these have been offset mainly by fewer orders in Australia, Cyprus, Iraq, and Kuwait. Contracts valued at £29,000,000 were obtained in Canada in 1956–57 by fourteen British firms, compared with just over £20,000,000 by twenty firms in 1955–56. According to the Canadian Government, expenditure on new construction this year will be about $5,563,000,000, which is $300,000,000 above that in 1956. Expenditure on constructional work per head of population is £100 in the U.S.A., £30 in Great Britain, and £A38 (= £30 sterling) in Australia. Building costs rose last year by about 5 per cent. in Great Britain, by 12 per cent. in Italy, by 1 per cent. in Portugal, and by about 5 per cent. in the United States and Canada.

Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

The Builder and his Contract, by Rex Proctor, Technical College, Great George Gate, Doncaster. October 22. 7.15 p.m.

Practical Formwork Design and Construction for Concrete, by J. G. Richardson. Institute of Technology, Bradford. October 14. 7.15 p.m.

Lightweight Concrete, by W. Kinniburgh. South Berkshire College of Further Education, Newbury. October 15. 7 p.m. Public Library, Vernon Road, Scarborough. October 30. 7 p.m.

The R.I.B.A. Form of Contract by David Gardam. College of Art and Technology, Leicester. October 16. 7.15 p.m.

Surface Finishes of Concrete, by F. W. Ward. Crown & Anchor Hotel, Ipswich. October 16. 7.30 p.m.

Legal Obligations of Building Contractors, by John J. Clarke. College of Art, Derby. October 24. 7.15 p.m. Also at Technical College, Shrewsbury. October 30. 7.15 p.m.

Some Aspects of Building Contracts, by Norman P. Greig. Technical College (New Buildings), Walsall. October 24. 7.15 p.m.

An Introduction to Prestressed Concrete, by R. C. Blyth. Cleveland Scientific and Technical Institution, Middlesbrough. October 29. 7 p.m.

Essentials of Good Concreting, by E. E. H. Bate. Technical College, Lincoln. October 30. 7.15 p.m.

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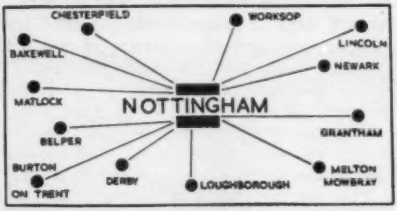
New Road in South Wales.

CONSTRUCTION of the second part of the Neath by-pass, from Earlswood to Lonlas, has now started. The road will be 3½ miles long with two carriageways 22 ft. wide separated by a strip 8 ft. wide and footpaths 6 ft. wide. About 300,000 cu. yd. of excavation will be necessary and the consolidated embankments will require some 500,000 cu. yd. of material, which is to be obtained from waste dumps in the neighbourhood.

The work includes seven bridges, namely a reinforced concrete bridge of 22 ft. span, a prestressed concrete bridge 140 ft. in length in three spans, a viaduct 680 ft. long in ten spans with a steel superstructure, a reinforced concrete deck, plain concrete abutments, and reinforced concrete piers on reinforced concrete piles, and three steel bridges. The total cost of the work, including acquisition of land, is estimated to be £1,742,000. The engineers are Messrs. Rendel, Palmer & Tritton. The main contractors are the Cleveland Bridge & Engineering Co., Ltd.



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FOR SALE. Wiedemann type R.4 hand-operated turret punching machine with twelve stations for sale. Steel plate construction. Fitted with punches and dies from ¼ in. to 1¼ in. diameter. Capacity: ¾ in. diameter, holes through ⅝ in. thick, or 1¼ in. diameter, through 14 s.w.g. Depth of gap to frame 24 in. Photo, etc., from F. J. EDWARDS LTD., 359 Euston Road, London, N.W.1.

FOR SALE. Melamine plastic-faced resin-bonded plywood, waterproof exterior grade, recommended for use in the construction of, or linings for, moulds for precast concrete. In thicknesses, ¼ in., ½ in., ¾ in. Full particulars from J. & T. T. BROWN LTD., Stockton Street, West Hartlepool.

FOR SALE. Steel tubes and fittings. Sections, strips, blanks, etc. List on request. E. STEPHENS & SON LTD., Bath Street, London, E.C.1. Clerkenwell 1731.

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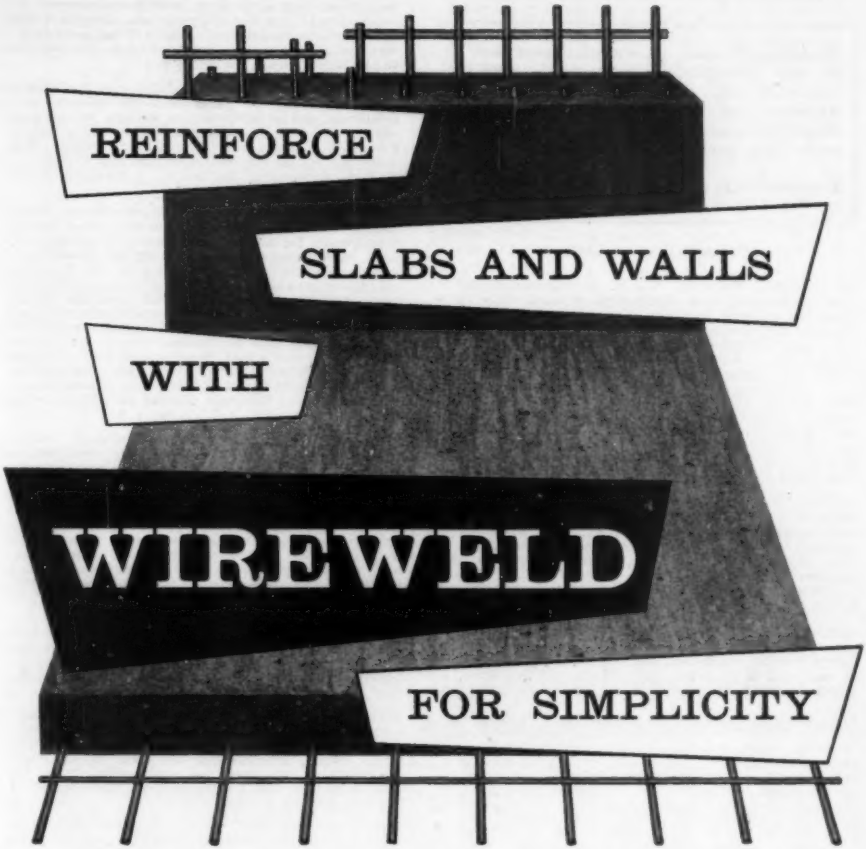
FOR HIRE. Lattice steel erection masts (light and heavy), 30 ft. to 150 ft. high, for immediate hire. BELLMAN'S, 21 Hobart House, Grosvenor Place, London, S.W.1



WIREWELD reinforcement is a high tensile steel fabric, manufactured from hard drawn steel wire electrically welded at all intersections to B.S. 1221, Part A. Made in oblong mesh and square mesh.

If you want to put steel into concrete, get in touch with

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MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 4d. a word : minimum, 7s. 6d. Situations Vacant, 5d. a word : minimum, 10s. Other miscellaneous advertisements, 5d. a word : minimum 10s. Displayed advertisements, 35s. per column inch. Box number 1s. extra.

Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT.

SITUATIONS VACANT. G.K.N. Reinforcements Ltd. have vacancies in their Birmingham drawing office for reinforced concrete designers and detailers. The office handles a wide variety of interesting structures. Five-days' week, pension scheme, staff canteen. Apply, giving details of experience, to Chief Engineer, G.K.N. Reinforcements Ltd., Alma Street, Smethwick, Staffs.

SITUATIONS VACANT. Detailer-draughtsmen and juniors required for varied reinforced concrete work. Five-days' week. Superannuation scheme. Moving shortly to new offices off Wandsworth Road (near Clapham Junction). Write, stating experience, age, and salary required, to The Rom River Co. Ltd., 90 Eversholt Street, London, N.W.1.

SITUATION VACANT. Civil engineering draughtsman required for work in London office of large civil engineering contractors. Work of varied nature, with scope for initiative and advancement. Commencing salary £600–£800 depending on experience. Five-days' week and luncheon vouchers. Pension scheme operates. Box 4228, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Consulting engineers have vacancies for civil and structural engineering assistants. Apply in writing, stating age, experience, and qualifications, to Botteley & Chaffer, Royal House, South Parade, Sutton Coldfield.

SITUATIONS VACANT. R. L. Bourqui & Partners, Thermix House, Kingston-upon-Thames, require experienced first-class detailers immediately for reinforced concrete work. Only experienced men with good printing and linework will be considered. Knowledge of design an advantage, but ability to detail from calculations essential. Should be able to construct general arrangement drawings of industrial structures, frame buildings, etc., from sketches. Above-average salaries offered to those considered to be both competent and conscientious. Telephone for appointment Elmbridge 0439 or Elmbridge 0888 (office hours) or Elmbridge 2654 (evenings).

SITUATIONS VACANT. The Glasgow office of G.K.N. Reinforcements Ltd. require reinforced concrete designers and detailers in their new drawing office. Five-days' week. Pension scheme. Luncheon vouchers. Varied and interesting work with good salaries for the right men. Write, stating age and experience, to The Chief Engineer, 30 Pinkston Road, Glasgow, C.4.

SITUATIONS VACANT. Reinforced concrete designers and detailers required by progressive civil engineering firm in North Midlands. Good prospects for the right men. Apply, giving details of experience, qualifications, and salary required, to Box 4451, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

THE EXPANDED METAL COMPANY LTD.

Reinforced concrete designers and detailers required for permanent positions in London, West Hartlepool, Birmingham, and Manchester. Five-days' week. Staff pension scheme, etc. Apply in writing, giving brief details of experience, and salary required, to the Chief Engineer, The Expanded Metal Co. Ltd., 16 Caxton Street, London, S.W.1.

SITUATION VACANT. Structural engineer with at least three years' design experience in reinforced concrete work required by London consulting engineer. Five-days' week, varied and interesting work. Apply, giving particulars, to Box 4454, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete designers required. Should be experienced in modern design techniques, and be accustomed to working without supervision. Knowledge of structural steelwork an advantage. Permanent positions, interesting and varied work. Apply in writing, stating age, experience, and salary required, to Z. Pick, Consulting Engineer, 42 Ferncroft Avenue, London, N.W.3.

SITUATIONS VACANT. Air Ministry Works Designs Branch requires in London structural engineering designer-draughtsmen for reinforced concrete or structural steelwork, with sound technical training and several years' varied experience in design-detailing of : (a) Reinforced concrete construction for all types of buildings, or (b) Steel frame sheds, warehouses, and similar buildings. Salaries up to £1015 per annum. Starting pay dependent upon age, qualifications, and experience. Long-term possibilities, with promotion and pensionable prospects. Five-days' week. Three weeks three days' leave a year. Normally natural born British subjects. Write, stating age, qualifications, employment details including type of work done, to any Employment Exchange, quoting Order No. Borough 601.

SITUATION VACANT. Senior reinforced concrete engineer-designer required by a large manufacturing organisation with branches throughout the United Kingdom. Must be conversant with Code of Practice, L.C.C. Bye-Laws, and able to design light frame structures and floor beams from estimating stage to final details. Liaison with production and sales departments of prime importance. First-class working conditions, pension scheme. Position calls for initiative and responsibility, and gives the right man a really good opportunity for progress. Salary according to experience and qualifications. Apply, giving full details and experience, etc., to Box A.542, Central News Ltd., 43 London Wall, London, E.C.2.

SITUATIONS VACANT. Peter Lind & Co. Ltd., Romney House, Tufton Street, Westminster, S.W.1, have vacancies for fully qualified site engineers for civil engineering contracts. Apply in writing only, stating age, experience, and salary required.

SITUATION VACANT. Consulting structural engineers require section leader able to design various reinforced concrete structures and to supervise detailing. Five-days' week. Holiday arrangements honoured. Apply, stating full details of experience, and salary required, to Box 4561, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Senior reinforced concrete designer required, fully experienced in the design of foundation work and structures for heavy industrial plants. High salary offered to competent applicant. Maximum assistance given in regard to housing. Grant towards removal expenses. Living allowance until removal effected. Pension and insurance scheme, technical library, sports club. Apply Ref. L, Staff Personnel Manager, Ashmore, Benson, Pease & Co., Stockton-on-Tees.

SITUATION VACANT. Engineering assistant, with experience of detailing reinforced concrete work, required to work with structural engineer on extensive and varied programme of multi-storey flats using unique constructional techniques. Salary £665–£1005 with limited placing for age. Superannuation. Applications, with particulars of experience, to Secretary, Scottish Special Housing Association Ltd., 14–21 Palmerston Place, Edinburgh 12, within 14 days of the appearance of this advertisement.

SITUATIONS VACANT. Experienced reinforced concrete designer-detailers required. Opportunity of contemporary design experience. Salaries up to £1100 according to experience. Five-days' week. John De Bremaeker & Partners, 3 Southampton Place, London, W.C.1. Chancery 3785.

SITUATION VACANT. Senior reinforced concrete designer required to take charge of a section of the London drawing office of a well-known firm of reinforced concrete engineers. Should be A.M.I.Struct.E. and/or A.M.I.C.E. Apply in writing, stating age, experience, and salary required, to Box 4468, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete designers and detailers required by London firm of consulting engineers. High salaries and good prospects. Telephone Tate 3611, or write Box 4467, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

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CIVIL AND STRUCTURAL DEPT. SENIOR ENGINEER, fully qualified, experienced in design and construction, investigations and research (or with a desire to specialise in such matters). This is a first-class position of considerable responsibility for a leading man.
The position offers outstanding opportunities for the successful applicant.
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Applications with full details of qualifications and experience to The Secretary, Terresearch Limited, Ruislip Road, Northolt, Greenford, Middx.

ATOMIC ENERGY

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require additional staff for their

NUCLEAR POWER DIVISION

in Erith and Mayfair.

There are vacancies for:

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(b) DRAWING OFFICE PROGRESS ENGINEER: This work will consist of checking drawings and progressing D.O. work. The minimum requirements for this post are H.N.C. (Civil Eng.) and sound D.O. experience. Ref. I.E. 18.

(c) DRAUGHTSMEN: Applicants should preferably hold an H.N.C. (Civil Eng.) and must have experience of civil engineering works, including reinforced concrete and structural steel. Ref. IE. 19.

Starting salaries are good. Prospects for good men are exceptional. These posts are permanent and pensionable. Working conditions are excellent. 5-days' week; 3 weeks' annual holiday. Write, quoting appropriate reference, to Staff & Training Division, Simon-Carves Ltd., Cheadle Heath, Stockport, Cheshire.

SITUATIONS VACANT. Local firm of civil engineering contractors specialising in riverside works and reinforced concrete structures in Northumberland and Durham, has vacancies for qualified and unqualified site engineers with previous site experience. Pension and bonus schemes in operation. Apply in writing, stating age, experience, and salary required, to The Secretary, Brims & Co., Ltd., Pandon Buildings, Newcastle-upon-Tyne, 1.

SITUATION VACANT. Designer with at least 4–5 years' experience in reinforced concrete required for London professional office. Knowledge of steelwork an advantage. Degree and/or professional qualification desirable. Must be capable of working with minimum supervision, and will be expected to attend progress meetings and deal directly with clients' representatives. The work is varied and offers scope for initiative. Five-days' week. Apply, stating full particulars, and salary required, to Farmer & Dark, Romney House, Tufton Street, London, S.W.1.

LONDON COUNTY COUNCIL

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There are vacancies for Reinforced Concrete Designers and Detailers at our Head Office and at most of our Design Offices in:
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Designs for a wide variety of interesting work, including prestressed and shell concrete, are prepared at all offices. Five-day week—Pension Scheme. Top rate salaries are paid to Designers, Designer-Detailers and Detailers in accordance with ability and experience. Openings available for Graduates and men with H.N.C. Apprenticeship scheme available for students of good educational standard.

THE BRITISH REINFORCED CONCRETE ENGINEERING CO. LTD., STAFFORD
Chief Engineer:
A. P. MASON, D.Sc., M.I.C.E., M.I.Struct.E.,M.A.S.C.E

H-W.843

Truscon

announce a gratifying response to recent staff advertisements — gratifying from the point of view of results—and gratifying to those who by writing to us, coming to see us, being offered and accepting well-paid positions, are now hard at work as members of the Truscon team. They seem to like it, and we're pleased to have them with us. We're still not exactly overloaded with staff, however, *and will always find room for a good man.* If you're feeling like a change or an opportunity to break new ground, drop a line to the Secretary. To-day, to-morrow, next month or next year—we're always pleased to hear.

The Trussed Concrete Steel Co. Limited.

35/41 Lower Marsh, London S.E.1

SITUATION VACANT. Designer-draughtsman for reinforced concrete required by Westminster consulting engineers. Varied work. Five-days' week. Luncheon vouchers. Salary according to experience. Full particulars to Box 4469, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

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MIDDLESEX COUNTY COUNCIL

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Should have sound knowledge of concrete technology, and ability and experience to direct labour engaged on large-scale manufacture under incentive bonus conditions.

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Residential travel and other travelling concessions available.

Superannuation scheme.

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Chief Civil Engineer,
Reference No. 48,
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5A, Euston Grove, LONDON, N.W.1

SITUATION WANTED.

SITUATION WANTED. Chartered structural engineer (M.I.Struct.E.) with fifteen years' experience. Chief Designer/General Manager specialising in production of large-span buildings framed in structural precast reinforced concrete seeks position of similar responsibility. Excellent references. Minimum salary £2000 per annum. Box 4466, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1

SERVICES OFFERED

SERVICES OFFERED. Services offered to consultants by free-lance engineers. Design and detailing of reinforced concrete and steel structures. Drawings, bending schedules, calculations. Telephone Silverthorn 8222. Box 4456, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

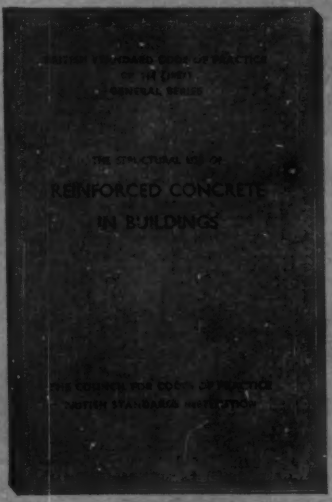
(*"For Sale" and "For Hire" advertisements on page 358.*)

TENTOR

REGISTERED TRADE MARK

and CP114 (1957)

Changes in the 1957 Edition of the British Standard Code of Practice, CP 114, make the use of the Tentor Bar, with its high tensile and bond strength, an even more attractive proposition.



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M-W.791

Published by CONCRETE PUBLICATIONS LTD., 14 Dartmouth Street, London, S.W.1. Telephone: Whitehall 4581
Printed by BUTLER & TANNER LTD., The Selwood Printing Works, Frome and London